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SIMULATION OF RUNOFF FROM AN AIR FORCE
BASE USING A PROGRAMMABLE CALCULATOR

James W. Luginbyhl, 2Lt, USAF

LSSR 60-81

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In this thesis the AFIT Runoff Model was developed and tested against other stormwater models and actual runoff data. The AFIT Runoff Model simulates the stormwater runoff from an Air Force Base. It was based on the Air Force Runoff Model (AFRUM), but it is used on a Texas Instruments TI-59 programmable calculator. The inputs to the model are land use characteristics of the watershed and an estimated Curve Number from the SCS Curve Number Model. The land use characteristics are percent impervious, percent forested, percent agricultural, percent denuded and surface area of the watershed.

SIMULATION OF RUNOFF FROM AN AIR FORCE

BASE USING A PROGRAMMABLE CALCULATOR

A Thesis

Presented to the Faculty of the School of Systems and Logistics

of the Air Force Institute of Technology

Air University

In Partial Fulfillment of the Requirements for the

Degree of Master of Science in Engineering Management

By

James W. Luginbyhl, BSCE
Second Lieutenant, USAF

September 1981

This thesis, written by

      Second Lieutenant James W. Luginbyhl

has been accepted by the undersigned on behalf of the faculty of the School of Systems and Logistics in partial fulfillment of the requirements for the degree of

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INTRODUCTION

Every year flash floods, produced by heavy rain-
fall, take lives and damage properties in many parts of the
world. It has been estimated that in the twenty years
preceding 1970, floods in Canada have caused a total of $100
million in damages with the cost to the federal government
of $40 million (6:1). In addition to the direct costs asso-
ciated with a flood are the loss of life, injury, incon-
venience and other indirect losses. Properly designed drain-
age structures could greatly reduce the amount of damage
produced by a flood.

A stormwater drainage structure conducts runoff
from places where it is not wanted to the nearest acceptable
discharge point or stores the runoff until it can be safely
released (1:41). Examples of these drainage structures
are culverts, storm sewers, drainage ditches and retention
basins.

The most important parameter in the design of a
stormwater drainage structure is an accurate estimate of the
rate of flow of the stormwater runoff the structure will be
expected to handle. Stormwater runoff is that portion of
the precipitation which flows over the ground surface during,
and for a short time after a storm (1:41).

There is a large cost penalty associated with the
inaccurate estimation of the runoff rate. An estimate that
is too low will result in a structure that cannot transport
or store the amount of runoff that could result from an
extreme rainfall event. Damages would result from flooding
from an underdesigned structure. The American Water Works
Association has reported that out of 293 dam failures in
the United States and other countries since 1799, about
twenty percent of the failures were due to underestimation
of the volume of water the spillway would need to transport
(6:1).

Costs resulting from overestimation of the rate
of flow of runoff are reflected in the increased costs of
construction for a larger structure. Drainage structures
are expensive. Approximately $500 million is spent annually
for highway culverts and small bridges in the United States
(2:1). That figure represents fifteen percent of the total
annual cost of interstate and state highways for construction
and maintainance (2:1). Larger structures cost much more
than smaller ones; therefore, each structure must be designed
so that it can safely carry the maximum amount of runoff
that is expected without having any excess capacity.

There are many models available for use when esti-
mating the runoff from a watershed. Some are simple enough
for hand calculations, while others are so complex that
they require a computer. Although the computer models are

generally much more accurate, the hand calculated methods are used more often (1:42). The more commonly used models are described in Chapter 2.

The Air Force Runoff Model, AFRUM, is a computerized model that has been developed to simulate runoff from an Air Force Base. However, this model has not been used extensively. Captain George W. Schlossnagle, the project officer for the development of AFRUM, believes that the reason the model has not been utilized to its fullest extent is a result of the difficulty involved in getting the program processed (14). Currently, an engineer who wants to use AFRUM must send all the necessary data to the Air Force Engineering and Services Center at Tyndall Air Force Base, Florida. Captain Schlossnagle said that in his opinion an engineer would rather use a hand calculated method, that is less accurate, because it takes less time and is easier than attempting to obtain results from a computerized model (14).

PURPOSE

The purpose of this thesis was to develop a stormwater runoff model, based on AFRUM, that could simulate runoff without the use of a large computer. The model that was developed is called the AFIT Runoff Model and operates on a programmable calculator. The AFIT Runoff Model eliminates the need for a large computer, while retaining the simulation methods found in AFRUM.

The AFIT Runoff Model was written for the Texas Instruments TI-59 was chosen because it is one of the most commonly used calculators that has the capacity to handle a program as complex as the AFIT Runoff Model. This calculator has up to 960 program steps available when no memories are used or up to 100 memories when only ten steps are used. The AFIT Runoff Model uses about 500 steps and 50 memories. The TI-59 has the ability to record the program on magnetic cards for easy reloading. Optional steps will be included for use when the calculator is locked on the PC-100A thermal printer, a TI-59 accessory that prints a hard copy.

The AFIT Runoff Model utilizes land use, soil type and hydrologic condition of the watershed to simulate runoff. The land use characteristics used are percent of the watershed that is impervious, percent forested, percent denuded and the surface drainage area in square miles. Soil type and hydrologic condition are input using the Curve Number of the Soil Conservation Service.

LIMITATIONS

The storms that can be used in the AFIT Runoff Model must be continuous, that is they cannot stop then start again. Another limitation to the model's use is the rate of rainfall cannot vary widely from one time period to another. These limitations must be made because

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of the method used in calculating the excess precipitation. Excess precipitation is the portion of the rain that contributes to the runoff. These two limitations would not be critical when using a design storm. The design storms, which the model was principally designed for, are not affected by these limitations. Most real storms can also be used with the AFIT Runoff Model; however, if a non-continuous storm or a storm whose rate of rainfall varies widely is encountered, AFRUM can be used instead.

VERIFICATION

The AFIT Runoff Model was tested by simulating the hydrographs for storms using the actual rainfall and runoff data. Hydrographs for the same storm events were also predicted by four other methods, the Environmental Protection Agency Stormwater Management Model (SWMM), the Army Corps of Engineers Urban Stormwater Runoff Model (STORM) and the Rational Method. The predicted hydrographs were statistically compared to the observed hydrograph and to each other. A goodness-of-fit test indicated the assumption of normal distributions was justified; therefore, parametric tests were used.

The rainfall and runoff data that were used came from three watersheds on Grissom Air Force Base in Indiana. The characteristics that were compared include: (1) peak runoff, (2) time to peak, and (3) total volume of the runoff.

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HYPOTHESES TESTED

For each of the three characteristics, two hypotheses were tested. The first test compared the mean of the characteristics predicted by the AFIT Runoff Model to the mean of the characteristics of the observed hydrograph to determine if they were statistically the same.

The second hypothesis tested whether the parameters predicted by each of the five models were significatly different.

RESEARCH QUESTIONS

The specific research questions addressed in this thesis were:

1. Can the AFIT Runoff Model accurately predict the runoff from an Air Force base?

2. Is the AFIT Runoff Model better than other commonly used stormwater runoff models?

Chapter 2

BACKGROUND

The estimation of runoff has been a problem
for engineers for many years. Because of the potential
dangers involved and the high cost of drainage structures,
many methods have been developed for predicting the amount
of runoff those structures will have to handle. However,
few of those models have been widely accepted. In the
report "Estimating Runoff Rates From Ungaged Small Rural
Watersheds", a research report sponsered by the American
Association of State Highway Officials, the authors
stated:

> One of the classical hydrologic problems yet
> unsolved is that of estimating floods of various
> frequencies from ungaged small rural watersheds....
> Many design engineers and hydrologists consider
> present methods as inadequate for estimating peak
> flow rates from ungaged small rural drainage basins.
> As a result there is no generally accepted design
> method. The plethora of methods being used throughout
> the United States and within individual states have
> produced inconsistent estimates of magnitude of
> floods of various frequencies [2:4].

This chapter will explain how stormwater models
are compared and then briefly describe some of the major
methods that are currently being used for the estimation
of stormwater runoff. The advantages and disadvantages
of each method will be discussed along with how AFRUM
compares to each method.

STORMWATER MODELS

The purpose of a stormwater model is to mathematically recreate a real world situation (7:129). Overton and Meadows classified modeling in three approaches: (1) deterministic, (2) parametric and (3) stochastic. In a deterministic system the output can be predicted for a given input, there is no element of chance involved. A stochastic model, on the other hand, has probability associated with the output. The difference between deterministic and parametric is a matter of degree. Overton and Meadows wrote:

> ... the parametric approach strives for the definition of the functional relations between hydrologic and geometric and land use characteristics of a catchment [11:159].

Stormwater models attempt to predict, at least, two major factors. The peak flow and the shape of the hydrograph are the most important parameters. Peak flow is the maximum rate of flow the drainage structure will be required to hold. For most purposes this value is all that is needed in the design of the drainage structure. The idea is that if the structure can handle the peak flow its capacity will not be exceeded. However, sometimes the peak will be maintained for only a short time; therefore, if some minor ponding can be allowed, the engineer may be able to design a smaller drainage structure. Before the engineer can make this kind of decision, or if the total volume of the runoff is needed the storm hydrograph is required.

HYDROGRAPH

A hydrograph is a chart plotting discharge against time (8:219). A typical hydrograph for a storm event is shown in Figure 1. The ordinate of a hydrograph is in cubic feet per second, or cubic meters per second and the abscissa is in units of time, hours for small basins or even days for large watersheds (17:112). The total volume of runoff is determined by the area under the curve.

The hydrograph usually has three general parts: (1) the rising limb or concentration curve, (2) the crest segment, and (3) the recession or falling limb (17:112). Each of these parts has certain inherent properties which, within limits, fix its shape (8:390). These sections are shown on Figure 1 along with the following three definitions. Lag time, is the time interval from the center of mass of the rainfall excess to the peak of the resulting hydrograph. Tp is the time to peak, which is the time interval from the start of rainfall excess, to the peak of the hydrograph. The final time interval shown on the hydrograph is the time of concentration, Tc, which is the time from the end of the rainfall excess to the point on the falling limb where the recession curve begins or the point of inflection (17:112).

RATIONAL METHOD

The most frequently used method for calculating peak runoff is the Rational Method (17:109). More than
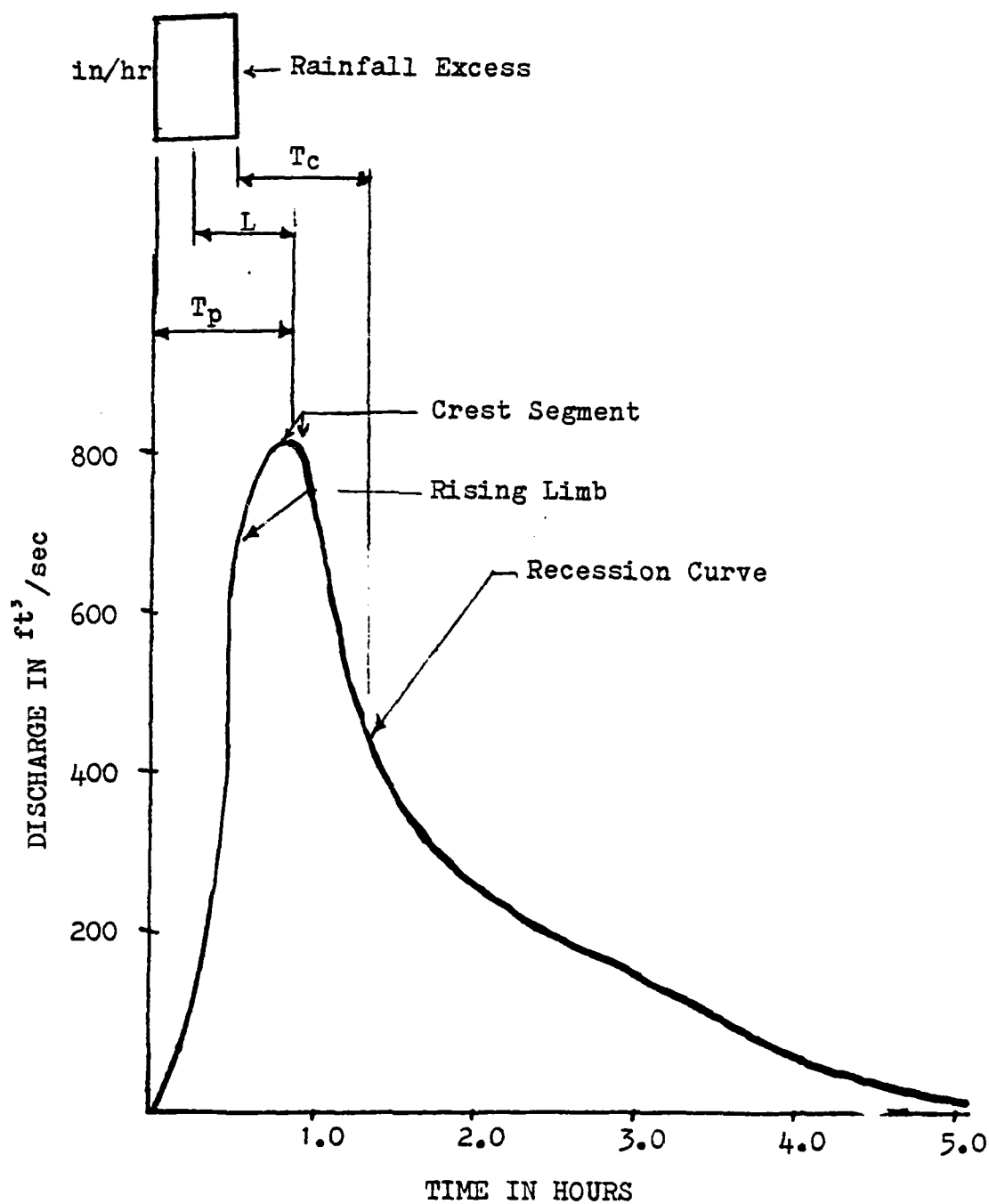
Figure 1. Typical Hydrograph (17:113)

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ninety percent of the engineering offices throughout the
United States answering a survey in 1956 on storm sewer
design practice used the rational method (1:42).

At the first Engineering Research Conference on
Urban Hydrology Research in 1965, it was pointed out that
the rational method still is widely used. Because of its
widespread use, the rational method is generally considered
current practice (1:42).

The rational method is a simple model that relates
runoff to rainfall intensity by the formula:

$$Q = CiA$$

in which Q is the peak runoff rate in cubic feet per second,
C is a runoff coefficient that depends on the characteristics
of the basin, i is the average rainfall intensity in inches
per hour, and A is the drainage area in acres (1:42).

The critical factor in this formula is the runoff
coefficient, C. It is usually estimated on the basis of
previous experience with similar areas and watersheds, since
it must represent many elements in runoff. It has to serve
for the following modifications: (1) infiltration losses,
(2) equilization of flow caused by surface detention,
(3) equilization of flow caused by valley and channel
storage and (4) the effects of the various physical factors
of the watershed on flow (3:343).

The peak flow computed by the rational method is
actually the peak of a equilateral triangle (17:119).
Figure 2 shows the hydrograph predicted by the rational

in/hr   Rainfall Excess

Tc

DISCHARGE IN ft^3/sec

Q_p

T_C T_C

TIME IN HOURS
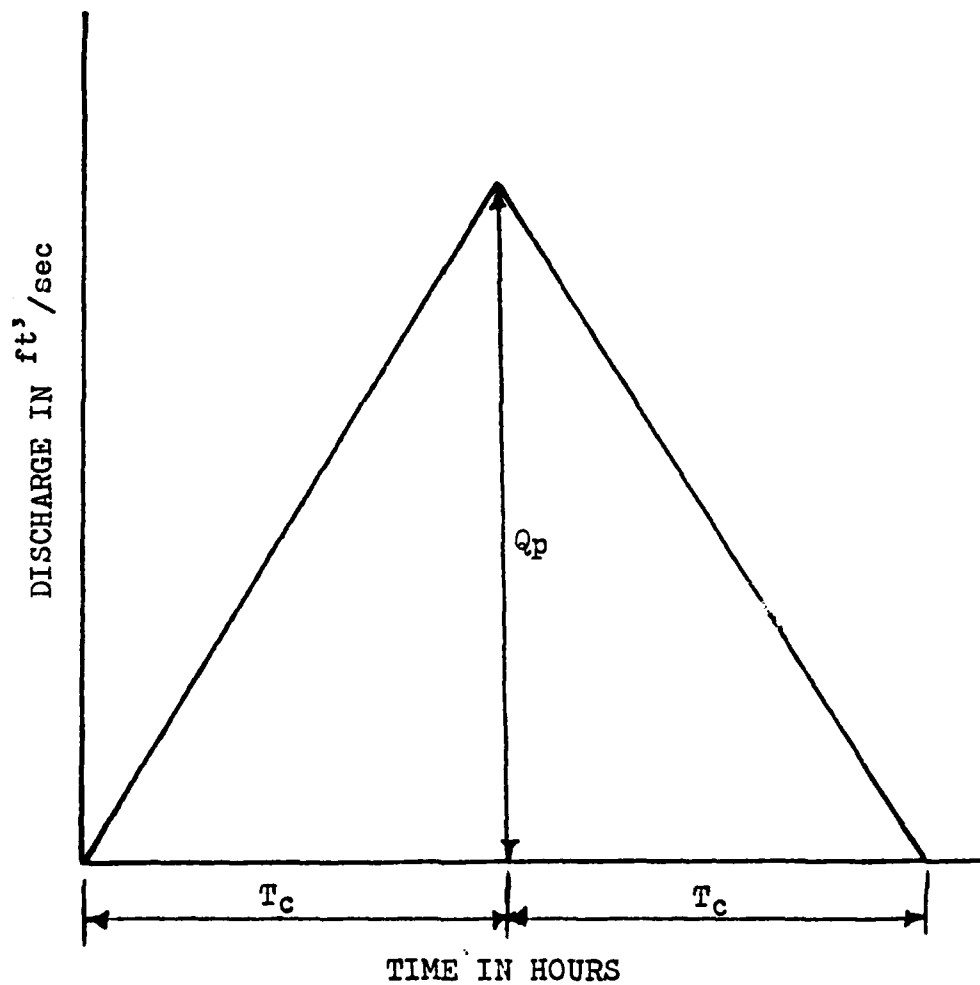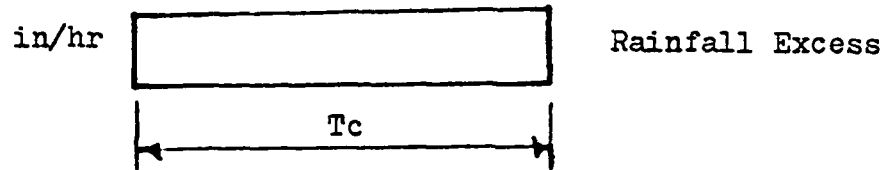
Figure 2. Hydrograph From Rational Method (17:119)

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method. The volume predicted by this hydrograph compared to
the volume predicted by the typical hydrograph shape in
Figure 1 indicates that the rational method is a conserva-
tive design procedure (17:120).

The advantages of computing a hydrograph by the
rational method hydrograph procedure are ease and simplicity.
The obvious disadvantage is inaccuracy. However, where small
watersheds, ten acres or less, are involved, this method
will produce satisfactory results (17:160). Edgar Foster
said:

> It is evident that successful use of this rational
> formula depends entirely upon the skill and judgement
> of the engineer in estimating suitable coefficients.
> It has been largely superseeded in estimation of flows
> for airport drainage by the more recent methods [3:343].

UNIT HYDROGRAPH METHOD

The next method for predicting the hydrograph from
rainfall is the unit hydrograph. The method was first
presented by Leroy K. Sherman in 932 and has been improved
and supplemented many times since then (10:514). This con-
cept has been called one of the most important contributions
ever made to the science of hydrology (18:247). The unit
hydrograph method is based on the idea that the physical
characteristics of the basin, such as shape, size and slope
are constant. Therefore, similarity in the shape of
hydrographs can be expected from storms of similar rainfall
characteristics (9:235).

A unit hydrograph is defined as the hydrograph produced

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by one inch of direct runoff from a storm of specified duration (9:238). The basic premise of the unit hydrograph method is that hydrograph produced by storms other than one inch of runoff are proportional in discharge throughout their length, and that when properly arranged with respect to time, the ordinates of several individual hydrographs can be added to give ordinates representing the total storm discharge (17:124).

The principles of this method are not rigorously true for all channels. Channel storage varies with stage; consquently, the unit hydrographs of large flows will differ from those of small flows (17:125). Commonly used unit hydrograph procedures have the tendency to compute peak flows that are higher than the actual runoff (17:147).

COMPUTERIZED MODELS

The computer has had a pronounced effect on storm-water modeling. Timothy Lazaro wrote:

> For some time now, the rational method and the unit hydrograph have been applied to estimate water quantity flows within the urban watershed. These procedures may be easily computed by hand. With the introduction of high speed analog and digital computers, a door has been opened into the use of formerly time-consuming mathematical methods. These methods allow significantly closer approximations of the physical processes of rainfall and runoff [7:154].

There are literally hundreds of computerized models that were developed to predict runoff. In the research report "Estimating Peak Runoff Rates From Small Ungaged Rural Watersheds", eighty-four sets of prediction

equations (2:19), including the methods used for highway construction by thirty-one states were compared with differing results (2:11).

Some models are highly specific in their application. They are only valid for one type of watershed, one region or even one watershed. The two most widely used computer models are the Envoronmental Protection Agency Stormwater Management Model, SWMM, and the United States Army Corps of Engineers Urban Stormwater Runoff Model, STORM.

COMPARISON OF STORMWATER MODELS

A two year study was performed at Grissom Air Force Base, Indiana to determine the effectiveness of the Air Force Runoff Model (15:1). In the study actual hydrographs from storms were determined for three watersheds. Also the hydrographs for those storms were simulated using AFRUM, SWMM, and STORM models. The predicted hydrographs from each model were compared with each other by how close they were to the observed hydrograph for peak discharge, time to peak, the volume of direct runoff and the shape of the hydrograph.

In comparing the three models it should be understood that every model is developed to fulfill specific objectives. These three models were not developed to fulfill the same objectives. Their structures and application procedures are different (12:139). Table 1 shows a comparison of the structural differences of the three models.

TABLE 1

SUMMARY OF MODEL COMPARISONS (12:157)

Model	Structure	Runoff Response	Parameter Prediction
AFRUM	Parametric	Nonlinear	Yes
STORM	Deterministic	Nonlinear	Default Values
SWMM	Parametric	Linear	Default Values

Because the STORM users manual did not include a prediction method for the model parameters, all input parameters were estimated by considering the examples in the manual. Error in these estimates might have contributed to the simulated peak discharge and volumes being higher than on the observed hydrographs and the shapes of the simulated hydrographs being only fair (12:129).

SWMM is a deterministic model; therefore, when input parameters are known with a high degree of cetainty, storm simulations should be modeled accurately. Unfortunately, the input parameters were not known. Because default values suggested in the users manual were used, the SWMM simulations were very poor (12:130).

AFRUM appeared to have done the best job of predicting the storm hydrographs in the Grissom study. AFRUM simulated high rainfall volume storms with a fair degree of accuracy but had trouble with the multi-burst and low intensity, short duration storms (12:137). It should be noted that for design purposes it is the storms that AFRUM simulated well, the high volume storms, that are used. These storms produce the highest peak flows, and if a structure will fail, it will do so during this type of storm.

Chapter 3

DESCRIPTION OF AFRUM

The Air Force Runoff Model is a parametric storm-water model that considers land use, soil type and hydrologic condition to predict runoff. Besides having a simulation phase, AFRUM also contains an analysis phase (12:139). AFRUM simulates direct runoff volume and rates using the United States Soil Conservation Service's Curve Number Model.

The parameters supplied as input to the model concerning land use are the percentage of the watershed that is impervious, percent denuded, percent forested and the total area of the drainage basin. All loses except evapotranspiration are lumped into a single initial abstraction.

DEVELOPMENT OF AFRUM

AFRUM was developed in the course of analysing 410 storms observed on 36 watersheds. The watersheds in the studies included Air Force Bases, agricultural lands, urban areas, forested and areas that were being strip mined for coal (13:2).

The model resulted from three separate studies that evaluated the effects of specialized land use on stormwater runoff. The United States Department of Energy was studying the effect of coal strip mining on runoff, while the Department of Interior Office of Water Resources Tech-

nology was investigating the effects of urbanization. The
last study contributing to AFRUM was conducted by the
Air Force on the runoff from Air Force bases (13:1).

AFRUM was developed for the Air Force by the
Department of Civil Engineering at the University of
Tennessee, Knoxville. The model was extensively modified
by the Air Force Engineering and Services Center (13:2).

DOUBLE TRIANGLE MODEL

AFRUM uses a Unit Response Function, URF, that was
coupled with the Curve Number Model to form the TVA double
triangle model. The quadrilateral URF that was formed, is
based on the concept that intial response from a watershed
comes from the riparian areas, or areas in and near the
water channel, and as the other areas become saturated they
too begin to contribute to runoff in the form of a delayed
response.

It is assumed that the two responses can be repre-
sented by two separate triangular response functions.
Both triangles begin at time zero but have different slopes.
When the two triangles are added together, they form the
quadrilateral unit response function.

The quadrilateral unit response function is shown
in Figure 3. The symbols being used in the Figure are listed
below.

I is the precipitation excess intensity in inches
per hour. I = 1/DT
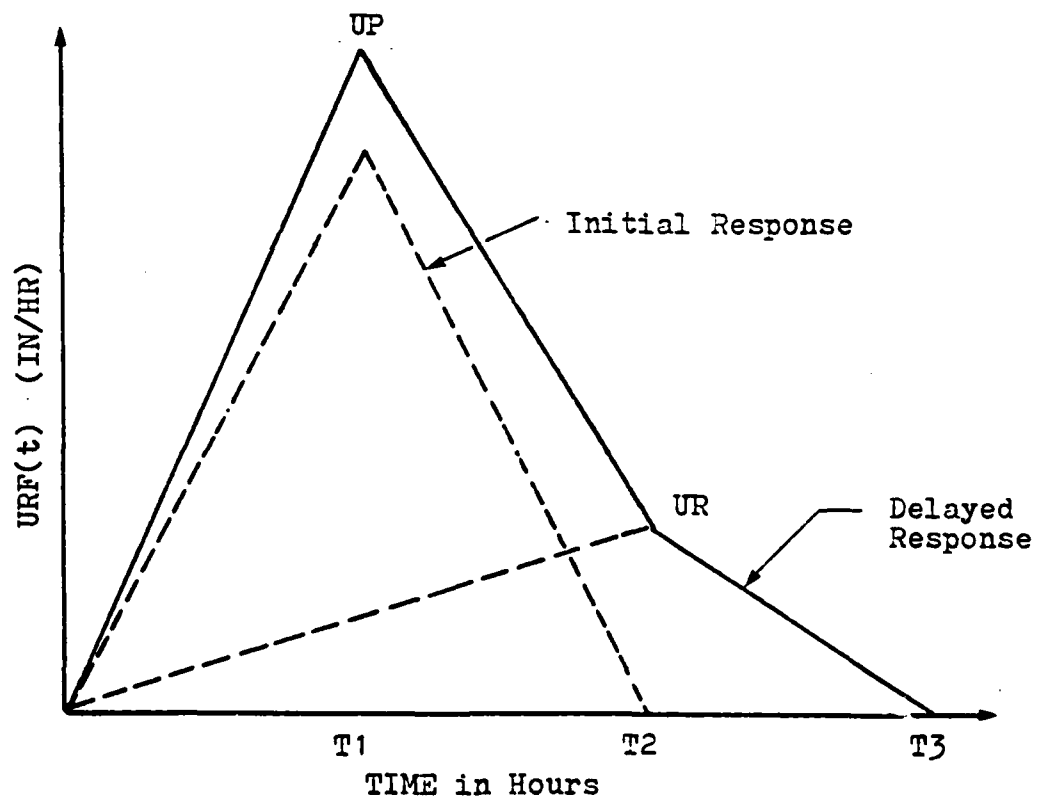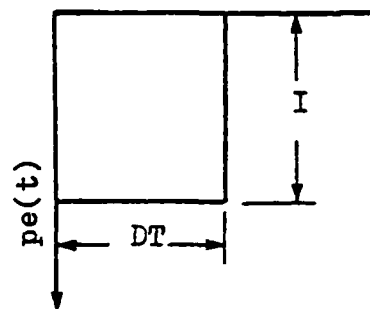
Figure 3. Double Triangle Model (13: 11).

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DT is the time interval used in abstracting rain-
fall and discharge records in hours.

UP is the peak of the unit response function.

UR is the peak of the delayed response function.

T1 is the time to peak of the initial response.

T2 is the time base of the initial response and
equal to the time to peak of the delayed response.

T3 is the time to the end of the delayed response.

pe(t) is the precipitation excess as a function of
time, t, in inches per hour.

URF(t) is the unit response function ordinate as
a function of time in inches per hour.

In deriving the URF, the peak of the delayed
response was assumed to occur at the end of the initial
response, and the time bases of both responses and the
time to peak of the initial response are integer multiples
of DT. The relative volumes in the initial and delayed
responses and the relative magnitudes of the peaks of the
individual responses were not fixed.

The double triangle URF is defined by the five
parameters UP, UR, T1, T2, and T3. T3 is found by the
equation:

$$T3 = (NOBS - NRAIN + 1) * DT$$

where NOBS is the number of storm hydrograph ordinates in
multiples of DT and NRAIN is the number of rainfall
increments in multiples of DT. By maintaining a unit

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volume, UR is calculated from:

$$UR = (2 - (UP * T2) / (T3 - T1))$$

Therefore, defining a storm URF involves determining vaues of ŪP, T1 and T2 (13:13).

The parameters UP, T1 and T2 are optimized using the pattern search technique. Since all five parameters describing the model are allowed to vary from storm to storm, the model is considered nonlinear (14:4).

NORMALIZED UNIT RESPONSE FUNCTION

The variability of the URF between storms within a watershed was explained by normalizing the time and discharge scale by the associated URF lag time, TL, where TL is equal to the time lapse between occurences of fifty percent of the rainfall excess block and fifty percent of the URF volume. The Normalized URF's are called NURF's (13:13).

The NURF for each land category, strip mined, 100 percent forested, urban without extensive storm sewers, urban with storm sewers, and agricultural land, has been determined empirically. The NURF's for each land use category is shown in Figure 4 along with the NURF observed for sheet surface runoff from a plane to provide a reference (13:13).

All of the NURF's in Figure 4 can be placed in the context of an initial response, IR, and delayed response, DR. The highest IR is from sheet surface runoff, and the lowest IR would be from a completely forested watershed. The initial

22
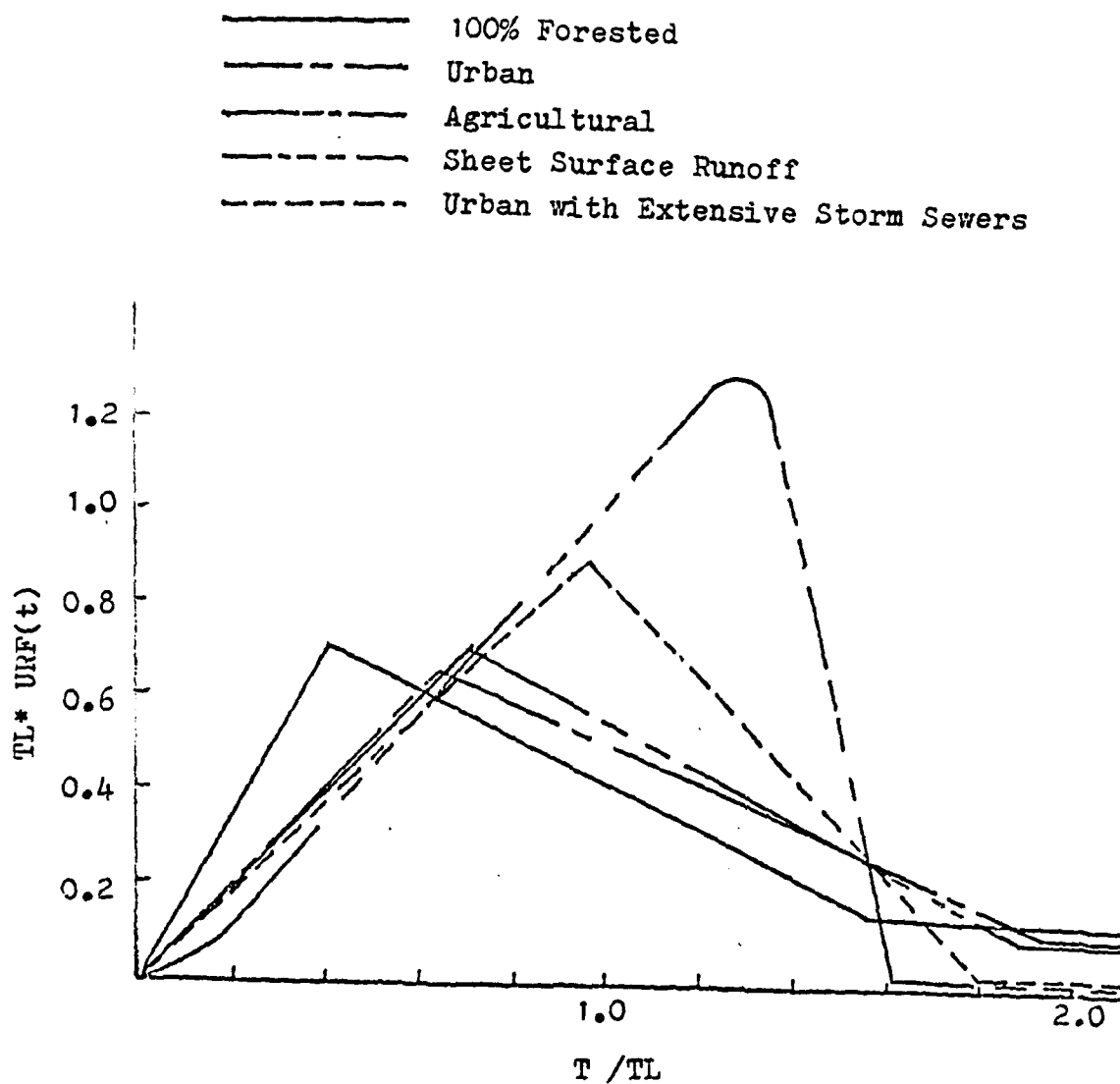
Figure 4. NURF for Various Land Use Conditions (13:14)

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and delayed responses are shown in Table 2 (11:74). The effect that storm sewers have on runoff is demonstrated in Table 2. Urban areas with extensive storm sewers have a higher percentage of the runoff in the initial response, which also means higher and quicker peak flows.

The AFIT Runoff Model was based on the Air Force Runoff Model; therefore, the theories discussed in this chapter apply to the AFIT Runoff Model as well.

TABLE 2

INITIAL AND DELAYED RESPONSE (13:78)

Land Use	IR (Percent)	DR (Percent)
Sheet Surface Runoff	97	3
Urban Storm Sewers	86	14
Urban	65	35
Agricultural	62	38
Contour Strip Mining	48	52
Forested	46	54

Chapter 4

METHODOLOGY

The AFIT Runoff Model is based on the Air Force
Runoff Model. AFRUM is a highly complex program consisting
of ten subroutines and over 1200 lines of input. The AFIT
Runoff Model is a simpler model developed for the TI-59
calculator. The TI-59 is a programmable calculator that
can hold a maximum of 960 steps. A program step is much
smaller than a line of computer input. One line from AFRUM
could require four to forty program steps to achieve the
same results.

Therefore, it is evident that some assumptions
and simplifications were needed to write the AFIT Runoff
Model. The assumptions made about the excess precipitation
resulted in the limitations described in Chapter 1, con-
tinuous and fairly uniform rainfall.

REDUCING THE MODEL

Even with these assumptions, the model is too long
to fit on a programmable calculator. Therefore, it must
be reduced by performing some of the functions manually.
Several one time calculations near the beginning and middle
of the program were selected for hand calculation, because
they were simple and quick. If they were calculated using
the program, they could use as much as twenty percent of the

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program steps.

In the model, there are several sets of empirical constants that depend on the characteristics of the watershed. These constants vary depending on whether the drainage area is: (1) urban or rural, (2) if it is agricultural, forested, or denuded and (3) if the watershed contains storm sewers. Rather than having these constants in the program, they will be set up in tables for the user to enter into the program as input.

PROGRAMMING THE CALCULATOR

A calculator program must be written differently than a program for a computer. In a computer program, all the values in each array can be calculated and stored at one time, then recalled as needed. A calculator program, on the other hand, cannot do this because of the limited storage space. The calculator program must go completely through to the end in one pass. Each time the calculator makes a pass through the program it will calculate another ordinate on the hydrograph.

The program was written following the procedures set forth in the TI59 user's manual. The program is divided into two major sections. The first section calculates the excess precipitation from the rainfall data. Some intermediate values are then calculated by hand and entered back into the program. The second section of the program

calculates the hydrograph ordinates so they can be plotted
with multiples of DT, the time interval between ordinates,
along the abscissa.

TESTING THE MODEL

To determine the accuracy and range of application
of the AFIT Runoff Model, the model was tested against the
observed hydrographs and hydrographs predicted by four
other models. Seventeen hydrographs from three watersheds
at Grissom Air Force Base were used for the comparisons.
The rainfall and runoff data for the hydrographs were col-
lected in 1978 and 1979. The three hydrograph parameters
used in the tests were peak flow, time to peak, and volume.

The first set of tests compared the hydrographs
predicted by the AFIT Runoff Model to the observed hydro-
graphs. The test used for the comparison was the matched
pair t-test. This test assumes both populations to be
distributed normally. The hypothesis tested whether
the mean of the observed population was significantly dif-
ferent from the mean of the predicted population.

$$H_0: \mu_{AFIT} = \mu_{observed}$$

$$H_1: \mu_{AFIT} \neq \mu_{observed}$$

The second set of tests compared the hydrograph
parameters predicted by the five models, the AFIT Runoff
Model, AFRUM, SWMM, STORM, and the Rational Method. The
mean of the parameters predicted by these models were
compared to each other and to the mean of the population of

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the observed parameters using a randomized complete block
design. This test also assumes normality. It compares all
the means together at one time in the following manner:

H_0: $\mu_{AFIT} = \mu_{AFRUM} = \mu_{RM} = \mu_{STORM} = \mu_{SWMM} = \mu_{observed}$

H_1: At least one inequality

If the null hypothesis, H_0, was rejected, another
test was performed. Duncan's multiple-range test determined
which of the six means were significantly different and
which ones were not.

The final statistical test used was the Lilliefors
goodness-of-fit test. This test determined that the assump-
tion of normality made in the other statistical tests was
justified.

Chapter 5

USING THE AFIT RUNOFF MODEL

This chapter explains the procedures and hand cal-
culations that are necessary to use the AFIT Runoff Model.
This chapter can serve as a user's guide for individuals
who wish to use the model to design a stormwater drainage
structure for an Air Force base.

Before these procedures can be used, the program
listed in Appendix A must be loaded into the calculator.
To load the program, the calculator is placed in the learn
mode by pressing the LRN key. After the program has been
entered, the LRN key must be pressed again to exit the
learn mode.

EXCESS PRECIPITATION

When rain strikes the earth's surface it can either
infiltrate into the soil, evaporate back to the atmosphere,
be retained in surface storage or flow over the surface.
Excess precipitation is the portion of the rainfall that
flows over the surface to become runoff. The first part
of the AFIT Runoff Model's program was designed to cal-
culate the excess precipitation based on the Soil Conserva-
tion Service Curve Number Model. Soils are divided into
four hydrologic soils groups: A, B, C, and D. Group A
soils have a high infiltration rate even when throughly wet.

When thoroughly wet, group B soils have a moderate infiltration rate, group C soils have a slow infiltration, group D soils have a very slow infiltration rate (16:181). More than 9,000 soils and their hydrologic group are listed in reference 16.

Rain that infiltrated into the soil from a previous storm and is still present in the soil is called antecedent moisture. In the Curve Number Model, Antecedent Moisture Condition II is an average condition while Condition III indicates that soils in the watershed are practically saturated from antecedent rain. Condition III has the highest runoff potential.

The curve number estimated from Table 3 was used in the AFIT Runoff Model to determine a surface retention factor by the formula:

$$S = (1000.0 / CN) - 10.0$$

where S = the effective surface retention

CN = the curve number

This factor was hand calculated and entered into the program by pressing the user key labled B.

LAG TIME

Lag time, TL, for a storm was simulated in the AFIT Runoff Model using the concept that varies inversely with the rainfall excess intensity. The lag time was calculated within the program using a factor called the lag modulus. The lag modulus, U, is empirically related to

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TABLE 3

RUNOFF CURVE NUMBERS FOR ANTECEDENT
MOISTURE CONDITIONS II AND III

Zoning Classification	Antecedent Moisture Conditions							
	II				III			
	Hydrologic Soil				Hydrologic Soil			
	A	B	C	D	A	B	C	D
Business, Industrial or Commercial	82	88	90	91	92	95	96	97
Apartment Houses	78	85	88	90	90	94	95	96
Schools	68	78	84	87	84	90	93	95
Urban Residential Lots ± 10,000 ft²	65	77	83	86	82	89	93	94
Suburban Residential Lots ± 12,000 ft²	62	76	82	85	80	89	92	94
Suburban Residential Lots ± 17,000 ft²	60	74	81	84	78	88	92	93
Suburban Residential	58	72	80	84	77	86	91	93
Parks and Cemetaries	55	71	79	83	74	86	91	93
Unimproved Areas	53	70	78	92	73	85	90	92
Lawns	45	65	75	80	66	82	88	91
Woods	36	60	73	79	*	*	*	*
Meadow (permanent)	30	58	71	78	*	*	*	*
Pasture or Range	49	69	79	84	*	*	*	*

*Data Unavailable

watershed characteristics.

For rural watersheds, the lag modulus is calculated
by the formula:

$$U = 0.060 * SQMI + 0.0203 * PF + 1.16$$

where SQMI = the area of the watershed in square miles

PF = the percent of the watershed that is forested
The lag modulus for urban watersheds can be calculated
from the formula:

$$U = 3.24 * (SQMI/PI)^{0.6}$$

where PI = the percent of the watershed that is impervious
After calculating the lag modulus, it was entered into the
program through user key A.

The time interval that was used to abstract the
hydrograph, DT, was entered through the user key C. DT can
be any time interval that is desired, generally it is in
tenths of a hour or fourths of an hour.

The cumulative rainfall for each DT was entered,
one at a time into user key D until all of the rainfall
data were entered. Lag time of the watershed was then
calculated by the program after the user key E was pressed.

DOUBLE TRIANGLE PARAMETERS

With the lag time of the watershed known, the
parameters of the TVA Double Triangle Model, UP, UR, T1, T2,
and T3, can be calculated. Empirically derived values were
used to find the parameters. These values depend on the

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land use characteristics of the watershed. The values for
watersheds that are urban without extensive storm sewers,
urban with storm sewers, denuded, agricultural and completely
forested are listed in Table 4. The formulas used to cal-
culate the double triangle parameters are:

$$UP' = KUP/TL$$

$$T1 = KT1 * TL$$

$$T2 = KT2 * TL$$

$$UR' = KUR/TL$$

$$T3 = T1 + ((2 - T2 * UP') / UR')$$

where KUP, KT1 KT2 and KUR are found in Table 4.

Before using those parameters, sveral adjustments
were made to them. The first adjustment was to round off
T1, T2 and T3 to the nearest multiple of DT. This was one
of the basic assumptions of the TVA Double Triangle Model,
it assures that the peak and inflection points occur at one
of the calculated hydrograph ordinates.

The next correction was to make sure that T2 was
at least one DT greater than T1 and that T3 was greater than
T2 by the same amount. This correction keeps the parameters
in their relative order. To keep T3 from being an extremely
large number, it was limited to fifteen times the value of
TL.

The final correction makes sure that the area of
the unit response function, URF, was equal to one. This
is accomplished by first calculating the area of the initial
URF from the formula:

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TABLE 4

DOUBLE TRIANGLE PARAMETER FACTORS

Land Use	KUP	KT1	KT2	KUR
Urban	0.663	0.632	1.88	0.12
Urban with storm sewers	0.900	0.956	1.80	0.035
Coal Strip Mined	0.740	0.253	1.085	0.21
Agricultural	0.705	0.695	1.87	0.13
Forested	0.716	0.394	1.57	0.10

$$AREA = (0.5 * T1 * UP') + (0.5 * (UP' + UR')$$
$$* (T2 - T1)) + (0.5 * UR' * (T3 - T2))$$

The area calculated was used to adjust UP and UR by the
formulas:

$$UP = (1.0/AREA) * UP'$$
$$UR = (1.0/AREA) * UR'$$

CALCULATING HYDROGRAPH ORDINATES

With the double triangle parameters known, the
hydrograph ordinates can be calculated. However, these
values were not entered directly into the program. The
number of ordinates in each section of the hydrograph
were calculated by the formulas:

$$N1 = T1/DT$$
$$N2 = T2/DT$$
$$N3 = T3/DT$$

where N1, N2 and N3 are the number of ordinates. It should
be noted that these three numbers should be whole numbers
due to the fact that T1, T2 and T3 were multiples of
DT. These numbers were entered into the program by pressing
user key B' while N1 is in the calculator display, then
N2 and N3 were entered by pressing the run/stop key, R/S,
for each in order.

The next values that were entered into the cal-
culator were calculated by the formulas:

$$S1 = UP/T1$$
$$S2 = (UR - UP)/(T2 - T1)$$

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$$S3 = (0 - UR)/(T3 - T2)$$

These values were entered by pressing user key C' with S1
in the calculator display, then entering S2 and S3 through
the R/S key.

One final entry was made before the program was
started. The surface area of the drainage basin in square
miles was entered through the user key A'.

After the program was finally loaded, the hydro-
graph ordinates were calculated by pressing user key D'.
Because a PC-100A printer was used for this thesis, each
ordinate was printed out without the program stopping.
However, if a printer was not available, the program
would have stopped after calculating each ordinate for the
user to record. The program would have to be restarted by
pressing the R/S key.

After all the ordinates were calculated, the
hydrograph was plotted by hand. Seventeen hydrographs
predicted by the AFIT Runoff Model are shown in Appendix G.
The observed hydrographs and the hydrographs predicted by
AFRUM, STORM and SWMM, in Appendix G, were obtained from
reference 12. The peak of the hydrograph predicted by the
Rational Method occurs at the end of the rainfall.

SUMMARY

This section summarizes all the formulas used in
the program and how the parameters are entered into the
calculator. The formulas were:

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$$S = (1000.0/CN) - 10.0$$

$$U = 3.24 * (SQMI/PI)^{0.6} \quad \text{for urban watersheds or}$$

$$U = 0.060 * SQMI + 0.0203 * PF + 1.16 \quad \text{for rural watersheds}$$

$$UP' = KUP/TL$$

$$T1 = KT1 * TL$$

$$T2 = KT2 * TL$$

$$UR' = KUR/TL$$

$$T3 = T1 + ((2 - T2 * UP') / UR')$$

T1, T2 and T3 must be corrected as specified in this chapter.

$$AREA = (0.5 * T1 * UP') + (0.5 * (UP' + UR') * (T2 - T1)$$
$$+ (0.5 * UR' * (T3 - T2))$$

$$UP = (1.0 / AREA) * UP'$$

$$UR = (1.0 / AREA) * UR'$$

$$N1 = T1/DT$$

$$N2 = T2/DT$$

$$N3 = T3/DT$$

$$S1 = UP/T1$$

$$S2 = (UR - UP) / (T2 - T1)$$

$$S3 = (0 - UR) / T3 - T2)$$

The data were entered by the following procedures:

Value in the Calculator Display	Calculator Key Pressed	Value Calculated by Program
S	B	
U	A	
DT	C	
Cumulative Rainfall	D	

Value in the Calculator Display	Calculator Key Pressed	Value Calculated by Program
Cumulative Rainfall	D	
•	•	
•	•	
•	•	
0	E	TL
N1	2nd B'	
N2	R/S	
N3	R/S	
S1	2nd C'	
S2	R/S	
S3	R/S	
SQMI	2nd A'	
0	2nd D'	Hydrograph Ordinate
	R/S	Hydrograph Ordinate
	R/S	Hydrograph Ordinate
	•	•
	•	•
	•	•

Chapter 6

STATISTICAL TESTING AND ANALYSIS

This chapter will detail the statistical tests used
to attempt to verify the research questions proposed at the
end of Chapter 1. Those questions again, were as follows:

1. Can the AFIT Runoff Model accurately predict
the runoff from an Air Force base?

2. Is the AFIT Runoff Model better than other
commonly used stormwater models?

The simulation results for each model, shown in
Appendix B, were used for the comparisons.

STATISTICAL TESTING

Throughout this chapter, several sets of hypotheses
will be presented in the general form:

$$H_0: \quad \text{Something will happen}$$
$$H_1: \quad \text{Something will not happen}$$
$$\text{alpha} = 0.05$$

The key to the test is the null hypotheis, H_0. The
objective of the test is to either reject H_0 or fail to
reject H_0. Notice that the acceptance of H_0 is not an alter-
native. If a test is performed and H_0 is rejected, then the
alternative hypothesis, H_1, is the choice to be selected.
If a test is performed and H_0 is not rejected, then the null
hypothesis is the alternative selected. The fact that H_0 is

not rejected does not in itself provide proof of the validity of H_0. It merely means that there is not enough statistical evidence available to reject H_0 (4:264).

The decision to reject or fail to reject H_0 is based on probabilities and not on certainty. Hence, there are chances of error in making a decicion. The value of alpha indicates the importance that is attached to the consequences associated with rejecting H_0 when, in fact, H_0 should not have been rejected. An alpha level of .05 means that a five percent chance of being wrong when H_0 is rejected can be accepted (5:199).

NORMAL DISTRIBUTIONS

Many statistical tests, including the tests used in this thesis, require the assumption that the samples being tested came from a normally distributed population. A goodness-of-fit test should be used to determine whether the assumption is justified or not justified. The test used in this thesis had the following hypotheses and alpha risk:

H_0: the probability distribution is Normal

H_1: the probability distribution is not Normal

alpha = .05

In this thesis, the Lilliefors goodnes-of-fit test was used. The test, shown in Appendix C, failed to reject H_0; therefore, the assumption of normality was justified.

RESEARCH QUESTION NUMBER 1

Can the AFIT Runoff Model accurately predict the

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runoff from an Air Force Base?

To test this question, the hydrograph predicted by the AFIT Runoff Model was compared to the observed hydrograph. Three hydrograph parameters were used in the comparison, peak runoff, time to peak and volume of runoff. Each parameter was tested using the matched pair t-test. This test uses the differences of the population means to make the inferences. The following equation is used in this test:

$$\mu_D = \mu_O - \mu_A$$

where μ_D = the mean of the population of the differences

μ_O = the mean of the population of the observed parameters

μ_A = the mean of the population of the parameter predicted by the AFIT Runoff Model

If the parameters predicted by the AFIT Runoff Model are from the same population as the observed parameters, then the two means will be equal and the mean of the differences will be zero. Thus, the hypotheses are:

H_0: $\mu_D = 0$

H_1: $\mu_D \neq 0$

alpha = .05

The results of the matched pair t-test for peak flow, time to peak and volume of runoff are shown in Appendix D.

The tests for peak flow and volume of runoff failed to reject H_0; therefore, it can be concluded that there is

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no significant difference between the peak flow and volume
predicted by the AFIT Runoff Model and the actual peak flow
and volume. In the test for the time to peak parameter, the
null hypothesis was rejected at an alpha level of 0.05.

RESEARCH QUESTION NUMBER 2

Is the AFIT Runoff Model better than other commonly
used stormwater models?

Four other models were compared to the AFIT Runoff
Model. The peak flow, time to peak and volume predicted by
the five models were all compared to the observed parameters
using the randomized complete block design analogue to the
paired t-test. The equation for this test is:

$$y = \mu + t + \beta + e$$

where y = the individual parameter

μ = the overall mean

t = a treatment, or model, effect

β = a block, or watershed, effect

e = a random error

If the parameter prediction from all of the stormwater models
and the observed parameter are statistically the same, the
treatment effect, t, would be equal to zero. Hence, the
hypothesis:

$$H_0: \quad t_1 = t_2 = t_3 = t_4 = t_5 = t_6 = 0$$
$$H_1: \quad \text{At least one inequality}$$
$$\text{alpha} = 0.05$$

The results of these tests, shown in Appedix E,

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were the same for all three parameters, reject H_0. This
means that at least one of the models was significantly
different from the others.

To determine which models are statistically related,
Duncan's multiple-range test was used. The results of this
series of tests are in Appendix F. No significant differ-
ence was found between the AFIT Runoff Model and the Air
Force Runoff Model in the prediction of the peak flow.
This can be expected seeing as the AFIT Runoff Model was
based on AFRUM. All the models, with the exception of
SWMM, produced peak flows that were statistically the same
as the actual peak flow.

In predicting the volume of the runoff, SWMM was
the only model, again, that predicted values that were
statistically different from the observed values. The AFIT
Runoff Model, AFRUM, STORM and the Rational Method were all
related to the observed volume.

Only the Rational Method predicted time to peak
values that were significantly different from the observed
values. All other models, the AFIT Runoff Model, AFRUM,
STORM and SWMM simulated values that were statistically
the same as the observed time to peak values according
to the Duncan's multiple range test. This is in contrast
to the results obtained from the matched pair t-test,
which concluded that the time to peak values predicted
by the AFIT Runoff Model were not the same as the

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observed values. The reason for the discrepancy is that statistics is not an exact science. There was a five percent chance of error when rejecting H_O. A random error caused one test to conclude one thing and another to contradict it.

Chapter 7

RECOMMENDATIONS AND CONCLUSIONS

The objective of this thesis was to develop a
stormwater runoff model that can accurately simulate the
runoff from an Air Force base without the aid of a computer.
The AFIT Runoff Model does not require a computer; all
that is needed to use this model is a Texas Instruments
TI-59 programmable calculator.

ACCURACY

The accuracy of the model was taken into consid-
eration in the first research question which stated: Can
the AFIT Runoff Model accurately predict the runoff from
an Air Force base? The statistical evidence tended to
support the conclusion that the AFIT Runoff Model can
predict the runoff accurately. The peak flow and volume
predicted by the AFIT Runoff Model were statistically the
same as the observed values. The AFIT Runoff Model did
not do as well predicting the time to peak, according to
the matched pair t-test.

The peak flow is all that is usually needed from
the model for design purposes. The time to peak and volume
parameters were tested because they are characteristics of
the hydrograph shape. The hydrograph is needed only when

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designing a retention or catch basin. All other structures
only need the peak flow for design.

COMPARISON TO OTHER MODELS

The second research question stated: Is the AFIT
Runoff Model better than other commonly used stormwater
models? The AFIT Runoff Model was compared to four other
commonly used models. The Rational Method is a non-
computerized method which is the most commonly used model.
The Environmental Protection Agency Stormwater Management
Model (SWMM), the Corps of Engineers Urban Stormwater
Runoff Model (STORM) and the Air Force Runoff Model (AFRUM)
are computerized models. All of the models were tested
against the AFIT Runoff Model and the observed hydrograph.

There was not enough evidence in this study to
prove that the AFIT Runoff Model is better at predicting
peak flow than the Rational Method. However, the Rational
Method has the tendency to overestimate the peak runoff .
from a high intensity, short duration storm, such as in
Appendix G-3.

The hydrographs produced by the AFIT Runoff Model
were - very close to those predicted by AFRUM. The AFIT
Runoff Model tended to peak lower and later than the hydro-
graphs predicted by AFRUM, but it was very similar in shape.
AFRUM has the advantage of automatically plotting the hydro-
graph and also predicting the quality of the runoff. The

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AFIT Runoff Model, on the other hand, has the advantages of being as accurate as AFRUM while not requiring a computer. The base level Civil Engineers can use the AFIT Runoff Model at their own base, a characteristic not found in AFRUM.

RECOMMENDATIONS

The AFIT Runoff Model was tested using three watersheds on the same Air Force base. Future research could center on testing the model at other bases in other regions of the United States or in other countries concentrating primarily on comparing the AFIT Runoff Model with the Rational Method. Attempts to improve the accuracy of the model should concentrate on improving the accuracy of the models that the AFIT Runoff Model is based on. The accuracy of the AFIT Runoff Model is directly related to the accuracy of the curve numbers from the SCS Curve Number Model and AFRUM.

The AFIT Runoff Model works on a Texas Instruments calculator; however, this model could be easily modified to operate on other programmable calculators. The Hewlett and Packard company manufactures a line of popular programmable calculators that could handle the AFIT Runoff Model. The conversion of the model to one of the other calculators would be as simple as changing a Fortran program to Basic.

SAFETY FACTOR

The final recommendation is directed to the engineer using the AFIT Runoff Model. Generally the safety factors used in the design of stormwater drainage structures are taken into account in the selection of the return period of the design storm. For example, a structure designed for a 100-year storm will be larger than a structure based on a twenty-five year storm. However, care should be taken in using the AFIT Runoff Model, if the engineer has been using a more conservative model such as the Rational Method. The Rational Method adds another safety factor by virtue of the conservative nature of the model. Because the AFIT Runoff Model simulates the runoff more closely to the actual runoff, the engineer may want to chose a design storm with a longer return period, such as using a 100-year storm rather than a 50-year storm. This may be more important if the area being drained by the structure can be severely damaged by the water if the structure fails.

CONCLUSION

The AFIT Runoff Model can be a useful tool for the Air Force engineer. It is simple to use yet as accurate as the more complicated computer models. The AFIT Runoff Model combines some of the advantages of a computer model with the advantages of the non-computerized

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methods. The Rational Method is easy to use and it does
not require a computer, but it may not always be as accurate
as the user would like. Computer models, like AFRUM, are
usur ly more accurate, but they require a computer to run.
The AFIT Runoff Model is easy to use, does not require a
computer and has simulation power similar to AFRUM.

The determination as to when the AFIT Runoff Model
should be used depends largely on the judgement of the
engineer. He may want to use the AFIT Runoff Model if
the size of the watershed indicates that a longer duration
storm would produce more runoff. The AFIT Runoff Model
should also be used if a hydrograph is desired. In
all other cases, the Rational Method could be used.

APPENDICES

APPENDIX A

LISTING OF THE AFIT RUNOFF MODEL

LISTING OF THE AFIT RUNOFF MODEL

Code No.	Key	Comments	Code No.	Key	Comments
00	2nd LBL	ENTER U	21	STO	
01	A		22	02	RAIN (I)
02	STO		23	−	
03	08	U	24	RCL	
04	R/S		25	03	RAIN (I−1)
05	2nd LBL	ENTER S	26	=	
06	B		27	STO	
07	STO		28	04	RF (I)
08	01	S	29	RCL	
09	R/S		30	02	RAIN (I)
10	2nd LBL	ENTER DT	31	STO	
11	C		32	03	RAIN (I−1)
12	STO		33	CLR	
13	00	DT	34	X⇆t	t = 0
14	20		35	RCL	
16	STO		36	04	RF (I)
17	18		37	2ndX≥t	
18	R/S		38	47	
19	2nd LBL	ENTER RAINFALL	40	RCL	
20	D		41	10	

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42	STO	
43	09	
44	GTO	
45	55	
47	RCL	
48	10	
49	+	
50	RCL	
51	04	
52	=	
53	STO	
54	09	ARF
55	RCL	
56	01	S
57	x	
58	0.2	
60	=	
61	X ⇌ t	t = .2 S
62	RCL	
63	09	ARF (I)
64	X ≥ t	
65	74	
67	RCL	
68	12	SRO (I-1)
69	STO	
70	11	SRO (I)

71	GTO	
72	98	
74	(	
75	RCL	
76	09	ARF (I)
77	-	
78	0.2	
80	x	
81	RCL	
82	01	S
83	)	
84	x²	
85	÷	
86	(	
87	RCL	
88	09	ARF (I)
89	+	
90	0.8	
92	x	
93	RCL	
94	01	S
95	=	
96	STO	
97	11	SRO (I)
98	-	
99	RCL	

100	12			126	2nd SUM	
101	=	PE (I)		127	05	
102	STO			128	RCL	
103	10			129	05	
104	SBR			130	R/S	
105	CE			131	2nd LBL	Calculate TL
106	RCL			132	E	
107	10			133	RCL	
108	÷			134	07	SSQPE
109	RCL			135	÷	
110	00			136	RCL	
111	=	PEI (I)		137	06	SUMPE
112	2nd SUM			138	=	REI
113	06	SUMPE		139	y^x	
114	X^2			140	0.4	
115	2nd SUM			142	=	
116	07	SSQPE		143	1/x	
117	RCL			144	x	
118	09			145	RCL	
119	STO			146	08	U
120	10			147	=	TL
121	RCL			148	2nd PRT	
122	11			149	R/S	
123	STO			150	2nd LBL	
124	12			151	CE	
125	1			152	CLR	

153	X ⇌ t t = 0	182	1
154	RCL	183	2nd SUM
155	10	184	18
156	X = t	185	CLR
157	193	186	STO
159	2nd SUM	187	12
160	19	188	STO
161	1	189	19
162	2nd SUM	190	GTO
163	12	191	196
164	4 *	193	1
165	X ⇌ t	194	2nd SUM
166	RCL	195	17
167	12	196	INV SBR
168	X ≥ t	197	2nd LBL ENTER SQMI
169	174	198	2nd A'
171	GTO	199	STO
172	196	200	02
174	RCL	201	20
175	19	203	STO
176	+	204	18
177	4 *	205	R/S
178	=	206	2nd LBL ENTER N!
179	STO IND	207	2nd B'
180	18	208	STO
181	CLR	209	06

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210	R/S	ENTER N2	236	RCL	
211	STO		237	14	
212	07		238	-	
213	R/S	ENTER N3	239	1	
214	STO		240	=	
215	08		241	STO	
216	R/S		242	05	
217	2nd LBL	ENTER S1	243	2nd DSZ	
218	2nd C'		244	05	
219	STO		245	250	
220	02		247	GTO	
221	R/S	ENTER S2	248	350	
222	STO		250	RCL	
223	03		251	14	
224	R/S	ENTER S3	252	X ⇄ t	
225	STO		253	RCL	
226	04		254	06	
227	R/S		255	X ≥ t	
228	2nd LBL	Calculate Hydrograph	256	322	
229	2nd D'		258	RCL	
230	2		259	07	
231	STO		260	X ≥ t	
232	16		261	297	
233	1		263	RCL	
234	2nd SUM		264	14	
235	14		265	-	

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266	RCL		292	STO
267	06		293	09
268	-		294	GTO
269	RCL		295	333
270	07		297	RCL
271	=		298	14
272	x		299	-
273	RCL		300	RCL
274	04		301	06
275	+		302	=
276	RCL		303	x
277	07		304	RCL
278	x		305	03
279	RCL		306	+
280	03		307	RCL
281	+		308	06
282	RCL		309	x
283	06		310	RCL
284	x		311	02
285	RCL		312	=
286	02		313	x
287	=		314	RCL
288	x		315	00
289	RCL		316	=
290	00		317	STO
291	=		318	09

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319	GTO	346	15
320	333	347	GTO
322	RCL	348	233
323	14	350	RCL
324	x	351	11
325	RCL	352	x
326	00	353	640
327	x	356	x
328	RCL	357	43560
329	02	362	+
330	=	363	(
331	STO	364	3600
332	09	368	x
333	SBR	369	12
334	1/x	371	=
335	RCL	372	x
336	09	373	RCL
337	x	374	15
338	RCL	375	=
339	10	376	R/S or PRT if using a printer
340	=	377	2nd D'
341	+	378	2nd LBL
342	RCL	379	1/x
343	15	380	RCL
344	=	381	17
345	STO	382	+

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383	1		405	$X \geq t$
384	=		406	415
385	$X \rightleftharpoons t$		408	RCL IND
386	RCL		409	18
387	16		410	STO
388	$X \geq t$		411	10
389	397		412	GTO
391	CLR		413	423
392	STO		415	4 *
393	10		416	2nd SUM
394	GTO		417	12
395	424		418	1
397	4 *		419	2nd SUM
398	STO		420	18
399	12		421	GTO
400	RCL		422	400
401	12		424	20
402	$X \rightleftharpoons t$		426	STO
403	RCL		427	18
404	16		428	INV SBR

* For extra long storms, these four numbers can be changed from four to five to avoid exceeding the calculators storage space.

APPENDIX B

SIMULATION RESULTS

APPENDIX B-1

PEAK FLOW SIMULATION RESULTS FOR MCDOWELL DITCH

Storm Date	Observed (cfs)	AFIT (cfs)	AFRUM (cfs)	STORM (cfs)	SWMM (cfs)	Rational Method (cfs)
3 Aug 78	15.0	14.9	18.6	58.2	8.1	17.0
27 Aug 78	6.7	2.6	2.7	7.2	10.5	3.8
15 Sept 78	6.6	4.1	4.7	7.6	55.4	91.0
14 Nov 78	13.0	8.1	9.0	30.0	74.6	11.8
3 Dec 78	22.0	25.1	42.2	55.8	74.1	13.7
11 Apr 79	0.9	1.3	2.4	6.3	12.9	5.4
8 Jul 79	9.2	12.1	17.5	66.4	225.0	14.2

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APPENDIX B-2

PEAK FLOW SIMULATION RESULTS FOR EAST DITCH

Storm Date	Observed (cfs)	AFIT (cfs)	AFRUM (cfs)	STORM (cfs)	SWMM (cfs)	Rational Method (cfs)
14 Nov 78	1.4	0.6	0.6	4.8	5.3	2.6
17 Nov 78	0.6	0.6	0.6	1.0	1.7	1.9
3 Dec 78	4.0	4.3	5.0	8.6	2.0	3.8
7 Dec 78	0.7	0.4	0.5	0.7	23.5	1.0
11 Apr 79	1.1	1.1	0.9	2.5	4.5	1.5
8 Jul 79	2.1	3.5	5.9	19.9	12.9	3.9

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APPENDIX B-3

PEAK FLOW SIMULATION RESULTS FOR CLINE DITCH

Storm Date	Observed (cfs)	AFIT (cfs)	AFRUM (cfs)	STORM (cfs)	SWMM (cfs)	Rational Method (cfs)
2 Aug 78	2.0	0.9	0.9	18.6	23.3	5.9
27 Aug 78	1.7	1.2	1.2	6.2	14.5	2.3
14 Sept 78	1.9	1.1	1.3	3.2	7.1	31.9
3 Dec 78	1.9	2.1	2.1	6.0	5.4	2.8

APPENDIX B-4

VOLUME SIMULATION RESULTS FOR MCDOWELL DITCH

Storm Date	Observed (in)	AFIT (in)	AFRUM (in)	STORM (in)	SWMM (in)	Rational Method (in)
3 Aug 78	0.166	0.159	0.178	0.451	0.110	0.458
27 Aug 78	0.066	0.065	0.067	0.059	0.080	0.165
15 Sept 78	0.052	0.049	0.054	0.058	0.150	0.163
14 Nov 78	0.309	0.313	0.320	0.485	0.440	0.488
3 Dec 78	0.581	0.564	0.568	0.484	0.510	0.502
11 Apr 79	0.112	0.110	0.112	0.180	0.275	0.349
8 Jul 79	0.568	0.571	0.568	0.950	4.410	0.918

APPENDIX B-5

VOLUME SIMULATION RESULTS FOR EAST DITCH

Storm Date	Observed (in)	AFIT (in)	AFRUM (in)	STORM (in)	SWMM (in)	Rational Method (in)
14 Nov 78	0.130	0.124	0.127	0.220	0.450	0.442
17 Nov 78	0.063	0.059	0.060	0.045	0.060	0.148
3 Dec 78	0.264	0.265	0.226	0.341	0.070	0.431
7 Dec 78	0.058	0.051	0.055	0.045	0.350	0.283
11 Apr 79	0.277	0.281	0.277	0.200	0.357	0.425
8 Jul 79	0.834	0.836	0.834	0.850	1.700	0.994

APPENDIX B-6

VOLUME SIMULATION RESULTS FOR CLINE DITCH

Storm Date	Observed (in)	AFIT (in)	AFRUM (in)	STORM (in)	SWMM (in)	Rational Method (in)
2 Aug 78	0.014	0.015	0.015	0.525	0.190	0.539
27 Aug 78	0.013	0.011	0.013	0.269	0.180	0.465
14 Sept 78	0.011	0.011	0.011	0.098	0.070	0.208
3 Dec 78	0.021	0.021	0.021	0.198	0.110	0.438

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APPENDIX B-7

TIME TO PEAK SIMULATION RESULTS FOR MCDOWELL DITCH

Storm Date	Observed (hrs)	AFIT (hrs)	AFRUM (hrs)	STORM (hrs)	SWMM (hrs)	Rational Method (hrs)
3 Aug 78	2.00	2.70	2.50	2.00	7.00	7.5
27 Aug 78	11.50	10.25	9.25	7.00	4.25	12.00
15 Sept 78	2.25	2.00	1.75	2.00	0.50	0.50
14 Nov 78	10.75	10.25	10.50	8.00	8.50	11.50
3 Dec 78	6.50	7.25	6.50	6.00	8.50	10.20
11 Apr 79	5.00	8.25	9.00	3.00	3.00	18.00
8 Jul 79	2.00	5.00	4.00	3.00	3.50	18.00

APPENDIX B-8

TIME TO PEAK SIMULATION RESULTS FOR EAST DITCH

Storm Date	Observed (hrs)	AFIT (hrs)	AFRUM (hrs)	STORM (hrs)	SWMM (hrs)	Rational Method (hrs)
14 Nov 78	10.50	11.25	11.00	9.00	8.75	12.00
17 Nov 78	7.50	7.75	7.50	6.00	4.50	5.50
3 Dec 78	4.75	4.75	4.75	7.00	3.25	8.00
7 Dec 78	7.25	9.25	9.00	4.00	6.25	14.50
11 Apr 79	3.00	13.50	12.00	5.00	3.00	20.00
8 Jul 79	3.00	6.50	5.00	3.00	1.50	18.00

APPENDIX B-9

TIME TO PEAK SIMULATION RESULTS FOR CLINE DITCH

Storm Date	Observed (hrs)	AFIT (hrs)	AFRUM (hrs)	STORM (hrs)	SWMM (hrs)	Rational Method (hrs)
2 Aug 78	0.75	1.50	1.25	2.00	1.00	7.00
27 Aug 78	2.25	2.50	2.25	2.00	0.75	15.5
14 Sept 78	1.25	1.25	1.00	2.00	1.00	0.50
3 Dec 78	5.75	6.00	5.50	4.00	4.00	12.00

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APPENDIX C

LILLIEFORS GOODNESS-OF-FIT TEST

LILLIEFORS TEST

H_0: f(x) is Normal

H_1: f(x) is not Normal

alpha = .05

$\bar{x}$ = 10.49

s = 6.854

x	$z = \dfrac{x - 10.49}{6.854}$		S_z	F_z	Max $\left\| F(z) - S(z) \right\|$
0.9	-1.396	0	0.143	0.0823	0.0607
6.6	-0.568	0.143	0.286	0.2843	0.1413
6.7	-0.553	0.286	0.428	0.2912	0.1368
9.2	-0.188	0.421	0.571	0.4247	0.1463 **
13.0	0.366	0.571	0.714	0.6443	0.0733
15.0	0.658	0.714	0.857	0.7454	0.1116
22.0	1.679	0.857	1.00	0.9535	0.0965

0.1463 < 0.300

FAIL TO REJECT H_0

Assumption of Normality is Justified

APPENDIX D

MATCHED PAIR T-TEST

APPENDIX D-1

PEAK FLOW T-TEST

H_o: $\mu_d = 0$

H_1: $\mu_d \neq 0$

Observed	AFIT Runoff Model	Difference
15.0	14.9	0.1
6.7	2.6	4.1
6.6	4.1	2.5
13.0	8.1	4.9
22.0	25.1	-3.1
0.9	1.3	-0.4
9.2	12.1	-2.9
1.38	0.61	0.77
0.59	0.58	0.01
4.03	3.71	0.32
0.65	0.40	0.25
1.05	1.11	-0.06
2.10	3.52	-1.42
2.02	0.88	1.14
1.70	1.18	0.52
1.90	1.07	0.83
1.90	2.07	-0.17
		7.41

$\bar{x} = 0.43588$

$S_x = 2.04756$

$t = \dfrac{\bar{x} - \mu_0}{S_x / \sqrt{n}} = \dfrac{0.43588 - 0}{2.04756 / \sqrt{17}} = 0.8777$

$1.746 > 0.8777$

FAIL TO REJECT H_0

There is no significant difference between the actual
peak flow and the peak flow predicted by the AFIT Runoff
Model.

APPENDIX D-2

VOLUME T-TEST

H_0: $\mu_D = 0$

H_1: $\mu_D \neq 0$

alpha = .05

Observed	AFIT Runoff Model	Difference
0.166	0.159	0.007
0.066	0.065	0.001
0.052	0.049	0.003
0.309	0.313	-0.004
0.581	0.564	0.017
0.112	0.110	0.002
0.568	0.571	-0.003
0.130	0.124	0.006
0.063	0.059	0.004
0.264	0.265	-0.001
0.058	0.051	0.007
0.277	0.281	-0.004
0.834	0.836	-0.002
0.014	0.015	-0.001
0.013	0.011	0.002
0.011	0.011	0
0.021	0.021	0

$S_x = 0.00522$

$\overline{X} = 0.002$

$$t = \frac{\overline{X} - \mu_o}{S_x / \sqrt{n}} = \frac{0.002 - 0}{0.00522 / \sqrt{17}} = 1.5797$$

$1.746 > 1.5797$

FAIL TO REJECT H_o

There is no significant difference between the observed

volume and the volume predicted by the AFIT Runoff Model.

APPENDIX D-3

TIME TO PEAK T-TEST

$H_o:$ $\mu_D = 0$

$H:$ $\mu_D \neq 0$

alpha $= .05$

Observed	AFIT Runoff Model	Difference
2.00	2.70	-0.70
11.50	10.25	1.25
2.25	2.00	0.25
10.75	10.25	0.50
6.50	7.25	-0.75
5.00	8.25	-3.25
2.00	5.00	-3.00
10.50	11.25	-3.00
7.50	7.75	-0.25
4.75	4.75	0
7.25	9.25	-2.0
3.00	13.5	-10.50
3.00	6.50	-3.50
0.75	1.50	0.75
2.25	2.50	-0.25
1.25	1.25	0
5.75	6.00	-0.25

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$S_{\bar{x}} = 2.74608$

$\bar{x} = -1.3206$

$$t = \frac{-1.3206 - 0}{2.74608 / \sqrt{17}} = -1.983$$

$-1.983 < -1.746$

REJECT H_O

At an alpha level of .05, there is a difference between the
observed time to peak and the time to peak predicted by the
AFIT Runoff Model; however, at an alpha level of .01 the
null hypothesis could not be rejected.

APPENDIX E

RANDOMIZED COMPLETE BLOCK DESIGN TEST

APPENDIX E-1

RANDOMIZED COMPLETE BLOCK DESIGN FOR PEAK FLOW

$$H_0: \quad t_1 = t_2 = t_3 = t_4 = t_5 = t_6 = 0$$
$$H_1: \quad \text{At least one inequality}$$

Observed	AFIT	AFRUM	RM	STORM	SWMM	Rk	$\overline{Y}$
15.0	14.9	18.6	17.0	58.2	8.1	88.8	14.8
-6.7	2.6	2.7	3.8	7.2	10.5	33.5	5.6
6.6	4.1	4.7	91.0	7.6	55.4	169.4	28.2
13.0	8.1	9.0	11.8	30.0	74.6	146.5	24.4
22.0	25.1	42.2	13.7	55.8	74.1	232.9	38.8
0.9	1.3	2.4	5.4	6.3	12.9	29.2	4.8
9.2	12.1	17.5	14.2	66.4	225.0	344.9	57.4
1.4	0.6	0.6	2.6	4.8	23.5	33.5	5.6
0.6	0.6	0.6	1.9	1.0	1.6	6.4	1.1
4.3	3.7	3.6	3.8	8.6	2.0	25.8	4.3
0.7	0.4	0.5	1.0	0.7	23.5	26.7	4.5
1.1	1.1	0.9	1.5	2.5	4.5	11.6	1.9
2.1	3.5	5.9	3.9	19.9	12.9	48.2	8.1
2.0	0.9	0.9	5.9	18.6	23.3	51.6	8.6
1.7	1.2	1.2	2.3	6.2	14.5	27.1	4.5
1.9	1.1	1.0	31.9	3.2	7.1	46.2	7.7
1.9	2.1	2.1	2.8	6.0	5.4	20.3	3.4
90.6	83.3	114.5	214.5	303.0	578.9	1342.1	
5.33	4.9	6.7	12.6	17.8	34.1		

1.0

2.8   2.5

3.2   2.2

2.0

1.1

1.8

1.25   1.4   1.6
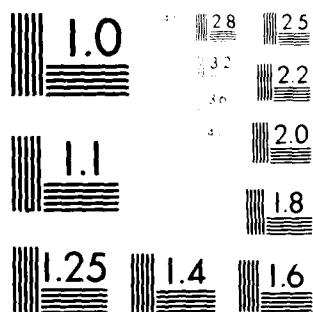
MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS

$$CF = \frac{(\Sigma\Sigma\Sigma Y_{ijk})^2}{JKN} = \frac{(1342.08)^2}{12 * 6 * 1} = 17658$$

$$SST = \Sigma\Sigma\Sigma \ Y^2_{ijk} - CF = 91787 - 17658 = 74128$$

$$SSC = \frac{\Sigma \ C_j^2}{KN} - CF = 11829$$

$$SSR = \frac{\Sigma \ R_k^2}{JN} - CF = 22978$$

$$SSE = SST - SSC - SSR = 39321$$

ANOVA TABLE

Source	SS	d.f.	MS	F = MS/MSE
Columns (Models)	11829	5	2365.8	4.81
Rows (Storms)	22978	16	1436.1	2.92
Error	39321	80	491.5	
Total	74128	101		

$$F_{.05,5,80} = 2.33$$

$2.33 < 4.81$

REJECT H_o

The models are different.

APPENDIX E-2

RANDOMIZED COMPLETE BLOCK DESIGN FOR VOLUME

H_0: $t_1 = t_2 = t_3 = t_4 = t_5 = t_6 = 0$

H_1: At least one inequality

Observed	AFIT	AFRUM	RM	STORM	SWMM	Rk	Y
0.17	0.16	0.18	0.46	0.45	0.11	1.52	0.254
0.07	0.07	0.07	0.16	0.06	0.08	0.50	0.084
0.05	0.05	0.05	0.16	0.06	0.15	0.53	0.088
0.31	0.31	0.32	0.49	0.49	0.44	2.35	0.393
0.58	0.56	0.57	0.50	0.49	0.51	3.21	0.535
0.11	0.11	C.11	0.35	0.18	0.28	1.14	0.190
0.57	0.57	0.57	0.92	0.95	4.41	7.99	1.331
0.13	0.12	0.13	0.44	0.22	0.45	1.49	0.249
0.6	0.6	0.6	0.15	0.05	0.06	0.44	0.073
0.26	0.26	0.27	0.43	0.34	0.07	1.64	0.273
0.06	0.05	0.06	0.28	0.04	0.35	0.84	0.140
0.28	0.28	0.28	0.42	0.20	0.35	1.81	0.302
0.83	0.84	0.83	0.99	0.85	1.70	6.05	1.008
0.01	0.02	0.02	0.54	0.53	0.19	0.82	0.136
0.01	0.01	0.01	0.46	0.27	0.18	0.95	0.158
0.01	0.01	0.01	0.21	0.10	0.07	0.41	0.068
0.02	0.02	0.02	0.44	0.20	0.11	0.81	0.135
3.55	3.51	3.55	7.42	5.46	9.51	32.48	
0.21	0.21	0.21	0.44	0.32	0.56		

$CF = 10.343$

$SST = 24.869$

$SSC = 2.158$

$SSR = 9.8997$

$SSE = 12.8091$

ANOVA TABLE

Source	SS	d.f.	MS	$F = MS/MSE$
Columns	2.1582	5	0.43164	2.696
Rows	9.8997	16	0.61873	3.864
Error	12.8091	80	0.16011	
Total	24.867	101		

$F_{.05,5,80} = 2.33$

$2.33 < 2.696$

REJECT H_o

All models are not the same

APPENDIX E-3

RANDOMIZED COMPLETE BLOCK DESIGN FOR TIME TO PEAK

$H_0: \quad t_1 = t_2 = t_3 = t_4 = t_5 = t_6 = 0$

$H_1: \quad$ At least one inequality

Observed	AFIT	AFRUM	RM	STORM	SWMM	Rk	$\overline{Y}$
2.00	2.75	2.50	7.5	2.0	7.00	23.75	3.95
11.50	10.25	9.25	12.00	7.00	4.25	54.25	9.04
2.25	2.00	1.75	0.50	2.00	0.50	9.00	1.50
10.75	10.25	10.50	11.50	8.00	8.50	59.50	9.92
6.50	7.25	6.50	10.20	6.00	8.50	44.95	7.49
5.00	8.25	9.00	18.00	3.00	3.00	46.25	7.71
2.00	5.00	4.00	18.00	3.00	3.50	35.50	5.92
10.50	11.25	11.00	12.00	9.00	8.75	62.50	10.42
7.50	7.75	7.50	5.50	6.00	4.50	38.75	6.46
4.75	4.75	4.75	8.00	7.00	3.25	32.50	5.42
7.25	9.25	9.00	14.50	4.00	6.25	50.25	8.38
3.00	13.5	12.00	20.00	5.00	3.00	56.50	9.42
3.00	6.50	5.00	18.00	3.00	1.50	37.00	6.17
0.75	1.5	1.25	7.00	2.00	1.00	13.50	2.25
2.25	2.50	2.25	15.50	2.00	0.75	25.25	4.21
1.25	1.25	1.00	0.50	2.00	1.00	7.00	1.17
5.75	6.00	5.50	12.00	4.00	4.00	37.25	6.21
86.00	110.00	102.75	190.70	75.00	69.25	633.70	
5.06	6.47	6.04	11.22	4.41	4.07		

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CF = 3937

SST = 5920

SSC = 2537

SSR = 2743

SSE = 640

ANOVA TABLE

Source	SS	d.f.	MS	F = MS/MSE
Columns	2537	5	507	63.4
Rows	2743	16	171	21.4
Error	640	80	8	
Total	5920	101		

$F_{.05, 5, 80} = 2.33$

$2.33 < 63.4$

REJECT H_o

All models are not the same.

APPENDIX F

DUNCAN'S MULTIPLE RANGE TEST

DUNCAN'S MULTIPLE RANGE FOR PEAK FLOW

Model	$\bar{x}$
AFIT	4.90
Observed	5.33
AFRUM	6.74
RM	12.62
STORM	17.82
SWMM	34.06

MSE $=$ 491.5   d.f. $=$ 80

$$S_{\bar{x}} = \sqrt{\frac{MSE}{n}} = \sqrt{\frac{491.5}{17}}$$

$S_{\bar{x}} =$ 5.377

p $=$	2	3	4	5	6
Table Range	2.819	2.966	3.063	3.134	3.189
Least Significant Range	15.16	15.95	16.47	16.85	17.15

SWMM vs. AFIT	34.06 $-$ 4.9 $=$ 29.16 $>$ 17.15
SWMM vs. Observed	34.06 $-$ 5.33 $=$ 28.73 $>$ 16.85
SWMM vs. AFRUM	34.06 $-$ 6.74 $=$ 27.33 $>$ 16.47
SWMM vs. RM	34.06 $-$ 12.62 $=$ 21.44 $>$ 15.95
SWMM vs. STORM	34.06 $-$ 17.82 $=$ 16.24 $>$ 15.16

STORM vs. AFIT   17.82 -   4.90 = 12.92 < 16.85

STORM vs. Observed   17.82 -   5.33 = 12.49 < 16.47

STORM vs. AFRUM   17.82 - 6.75 = 11.08 < 15.95

STORM vs. RM   17.82 - 12.62 = 5.30 < 15.16

RM vs. AFIT   12.62 - 4.90 = 7.72 < 16.47

RM vs. Observed   12.62 - 5.33 = 7.29 < 15.95

RM vs. AFRUM   12.62 - 6.74 = 5.89 < 15.16

AFRUM vs. AFIT   6.74 - 4.90 = 1.84 < 15.95

AFRUM vs. Observed   6.74 - 5.33 = 1.41 < 15.16

Obseved vs. AFIT   5.33 - 4.90 = 0.43 < 15.16

AFIT	Observed	AFRUM	RM	STORM	SWMM

All models underlined by the same line are not
significantly different. All models, with the exception
of SWMM are statistically the same.

89

DUNCAN'S MULTIPLE RANGE TEST FOR VOLUME

Model	$\bar{x}$
AFIT	0.2026
Observed	0.2082
AFRUM	0.2086
STORM	0.3211
RM	0.4362
SWMM	0.5592

MSE $=$ 0.16011

$S_{\bar{x}}$ $=$ 0.09705

p $=$	2	3	4	5	6
Table Range	2.819	2.966	3.063	3.134	3.189
LSR	0.274	0.288	0.297	0.304	0.309

SWMM vs. AFIT	$0.5529 - 0.2062 = 0.353 < 0.$
SWMM vs. Observed	$0.5529 - 0.2082 = 0.351 > 0.$
SWMM vs. AFRUM	$0.5529 - 0.2086 = 0.351 > 0.$
SWMM vs. STORM	$0.5529 - 0.3211 = 0.238 > 0.$
SWMM vs. RM	$0.5529 - 0.4362 = 0.123 > 0.$

RM vs. AFIT   0.4362 – 0.2062 ≈ 0.230 < 0.304

RM vs. Observed   0.4362 – 0.2082 ≈ 0.228 < 0.297

RM vs. AFRUM   0.4362 – 0.2086 ≈ 0.228 < 0.288

RM vs. STORM   0.4362 – 0.3211 ≈ 0.115 < 0.274

STORM vs. AFIT   0.3211 – 0.2062 ≈ 0.115 < 0.297

STORM vs. Observed   0.321 – 0.2082 ≈ 0.113 < 0.288

STORM vs. AFRUM   0.3211 – 0.2086 ≈ 0.113 < 0.274

AFRUM vs. AFIT   0.2086 – 0.2062 ≈ 0.002 < 0.288

AFRUM vs. Observed   0.2086 – 0.2082 ≈ 0.0004 < 0.274

Observed vs. AFIT   0.2082 – 0.2062 ≈ 0.002 < 0.274

SWMM	RM	STORM	AFRUM	Observed	AFIT

The underlined models are statistically the same. SWMM
is statistically the same as the Rational Method, STORM
and AFRUM but not the AFIT Runoff Model or the Observed
hydrograph. All other models are related.

APPENDIX F-3

DUNCAN'S MULTIPLE RANGE TEST FOR TIME TO PEAK

Model	$\bar{x}$
SWMM	4.07
STORM	4.41
Observed	5.06
AFRUM	6.04
AFIT	6.47
Rational Method	11.22

$MSE = 8$

$S_{\bar{x}} = 0.686$

p =	2	3	4	5	6
Table Range	2.819	2.966	3.063	3.134	3.189
LSR	1.934	2.035	2.101	2.150	2.188

RM vs. SWMM	$11.22 - 4.07 = 7.15 > 2.188$
RM vs. STORM	$11.22 - 4.41 = 6.81 > 2.150$
RM vs. Observed	$11.22 - 5.06 = 6.16 > 2.101$
RM vs. AFRUM	$11.22 - 6.04 = 5.18 > 2.035$
RM vs. AFIT	$11.22 - 6.47 = 4.75 > 1.934$

AFIT vs. SWMM $6.47 - 4.07 \approx 2.40 > 2.150$

AFIT vs. STORM $6.47 - 4.41 \approx 2.06 < 2.101$

AFIT vs. Observed $6.47 - 5.06 \approx 1.41 < 2.035$

AFIT vs. AFRUM $6.47 - 6.04 \approx 0.98 < 1.934$

AFRUM vs. SWMM $6.04 - 4.07 \approx 1.97 < 2.101$

AFRUM vs. STORM $6.04 - 4.41 \approx 1.63 < 2.035$

AFRUM vs. Observed $6.04 - 5.06 \approx 0.98 < 1.934$

Observed vs. STORM $5.06 - 4.07 \approx 0.99 < 2.035$

Observed vs. SWMM $5.06 - 4.41 \approx 0.65 < 1.934$

STORM vs. SWMM $4.41 - 4.07 \approx 0.34 < 1.934$

RM   AFIT   AFRUM   OBSERVED   STORM   SWMM
        _____

        _____

Underlined models are the same. Only the Rational Method predicted values that were not the same as the observed values.

APPENDIX G
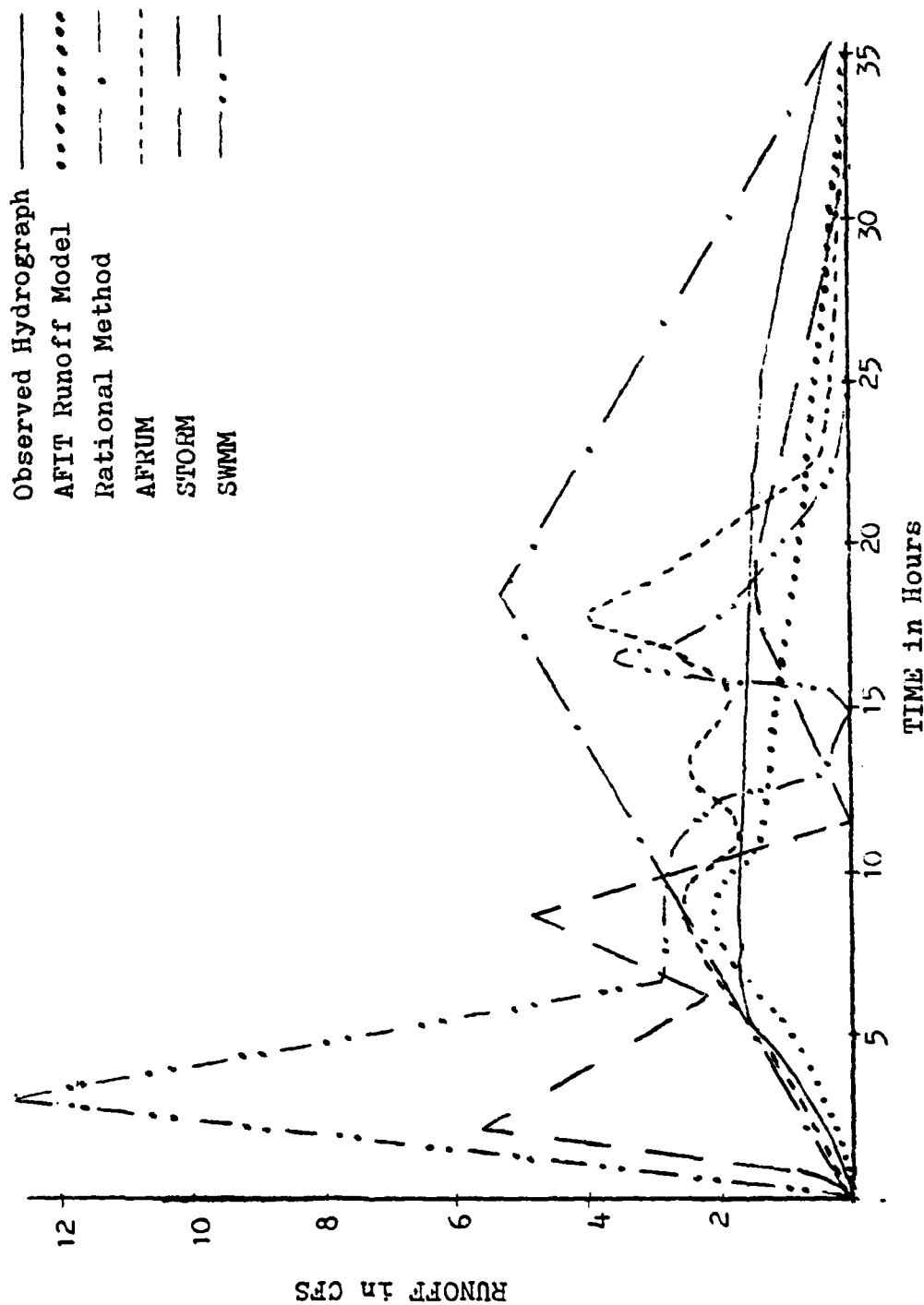
SIMULATED HYDROGRAPHS

G-1 MCDOWELL DITCH 3 August 1978

Observed Hydrograph
AFIT Runoff Model
Rational Method
AFRUM
STORM
SWMM

RUNOFF in CFS
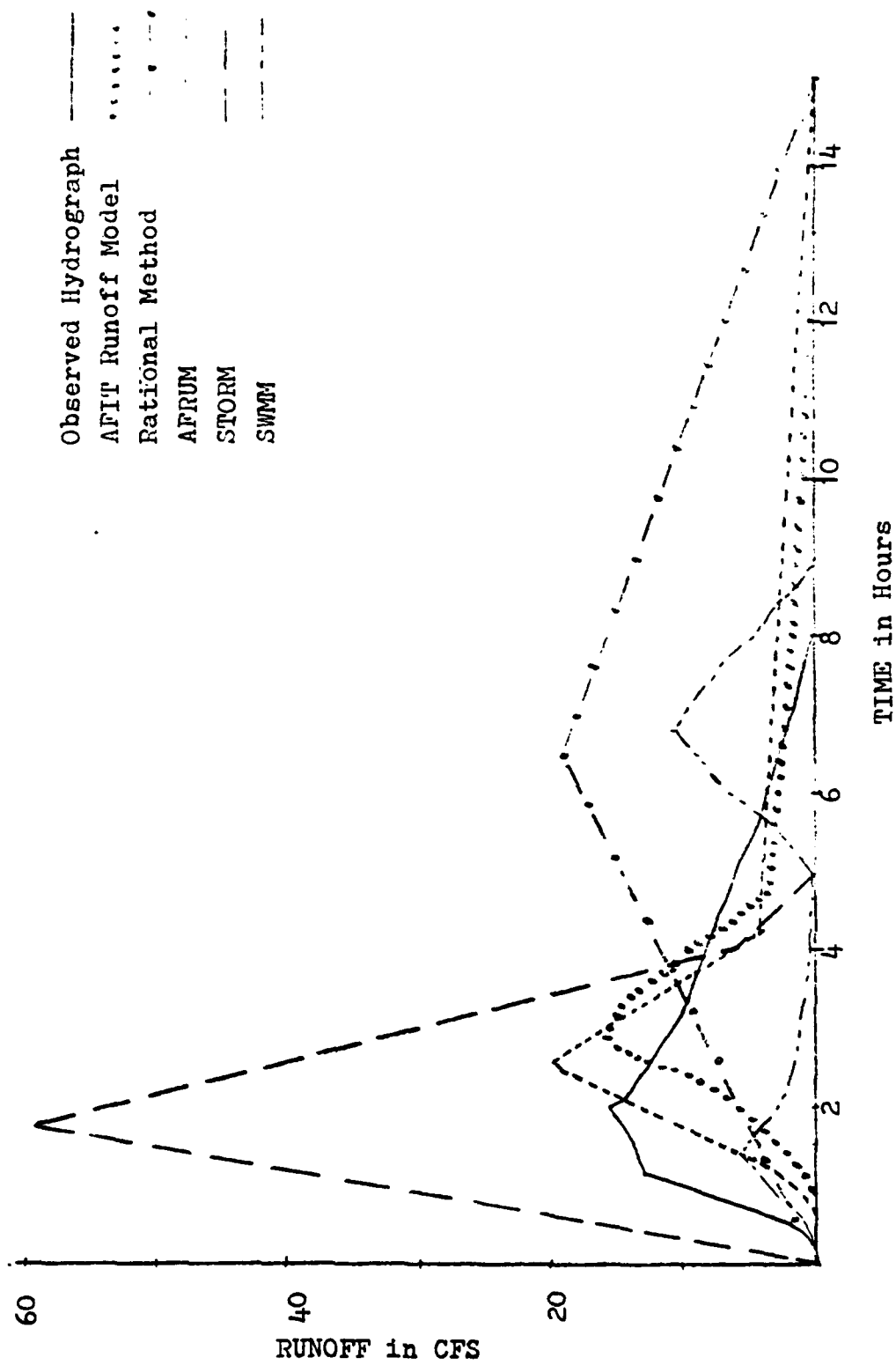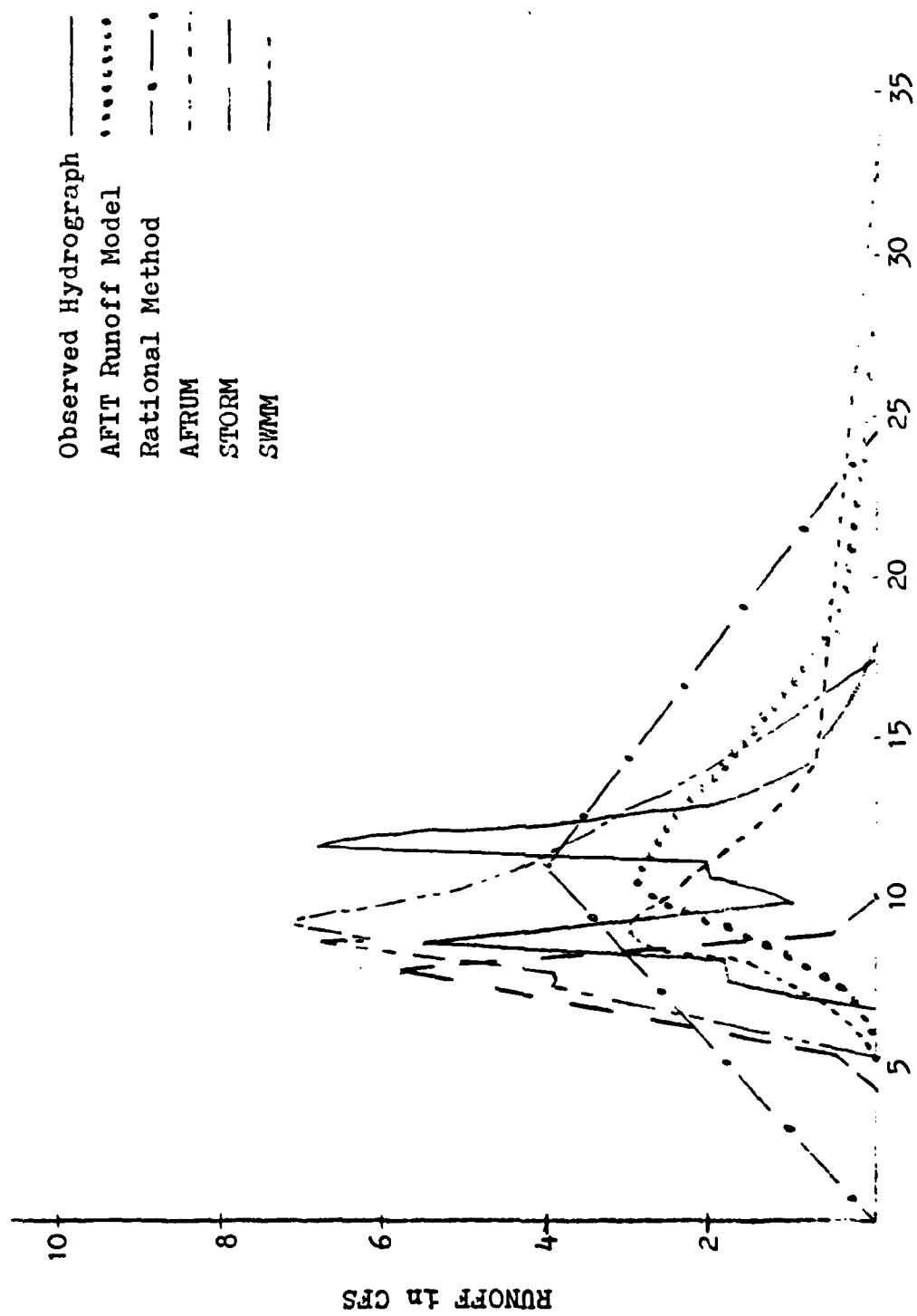
TIME in Hours

G-2   MCDOWELL DITCH   27 August 1978

Observed Hydrograph
AFIT Runoff Model
Rational Method
AFRUM
STORM
SWMM

$Q_p = 55$ cfs

$Q_p = 91$

RUNOFF in CFS
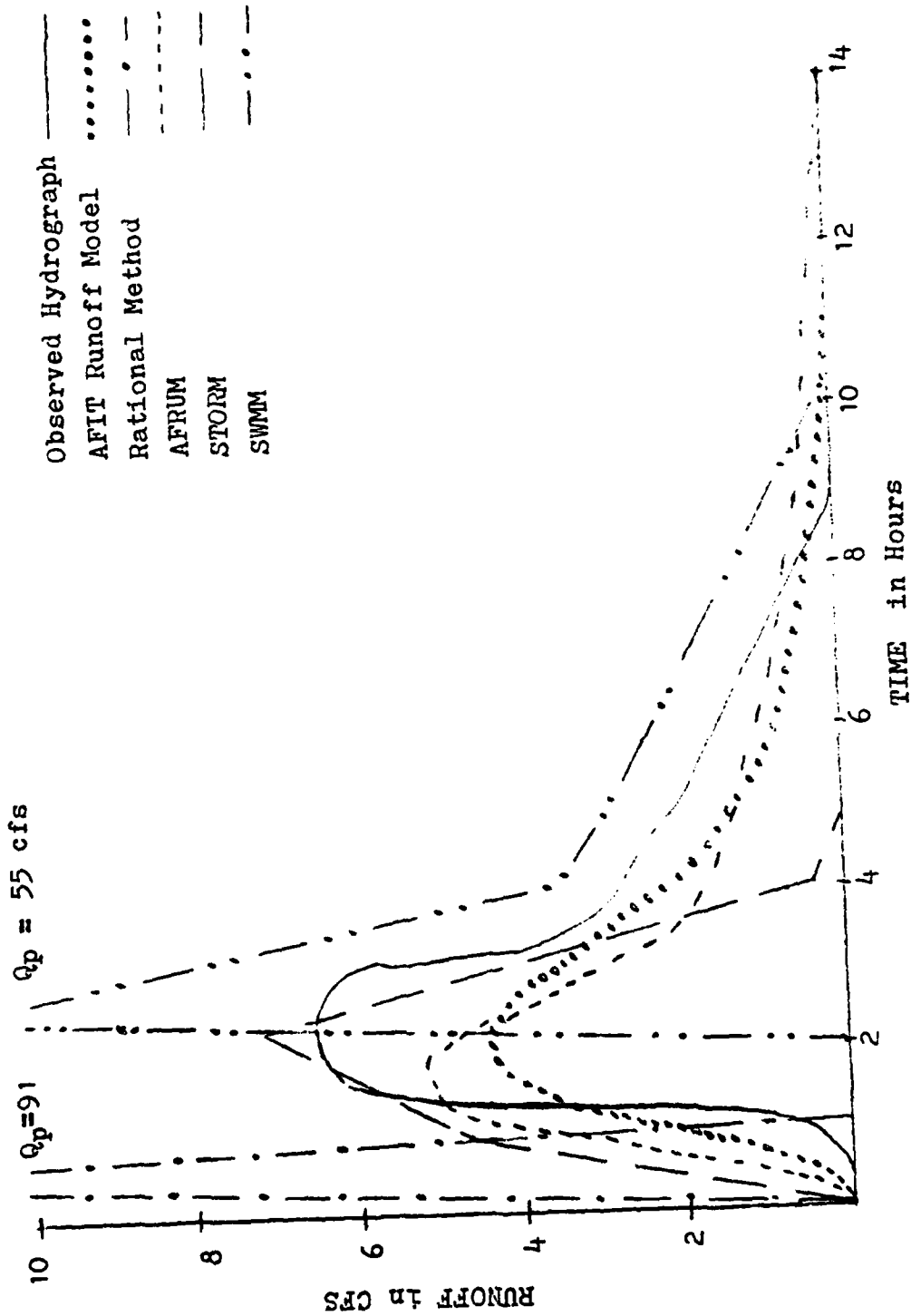
TIME in Hours

G-3   MCDOWELL DITCH   15 September 1978
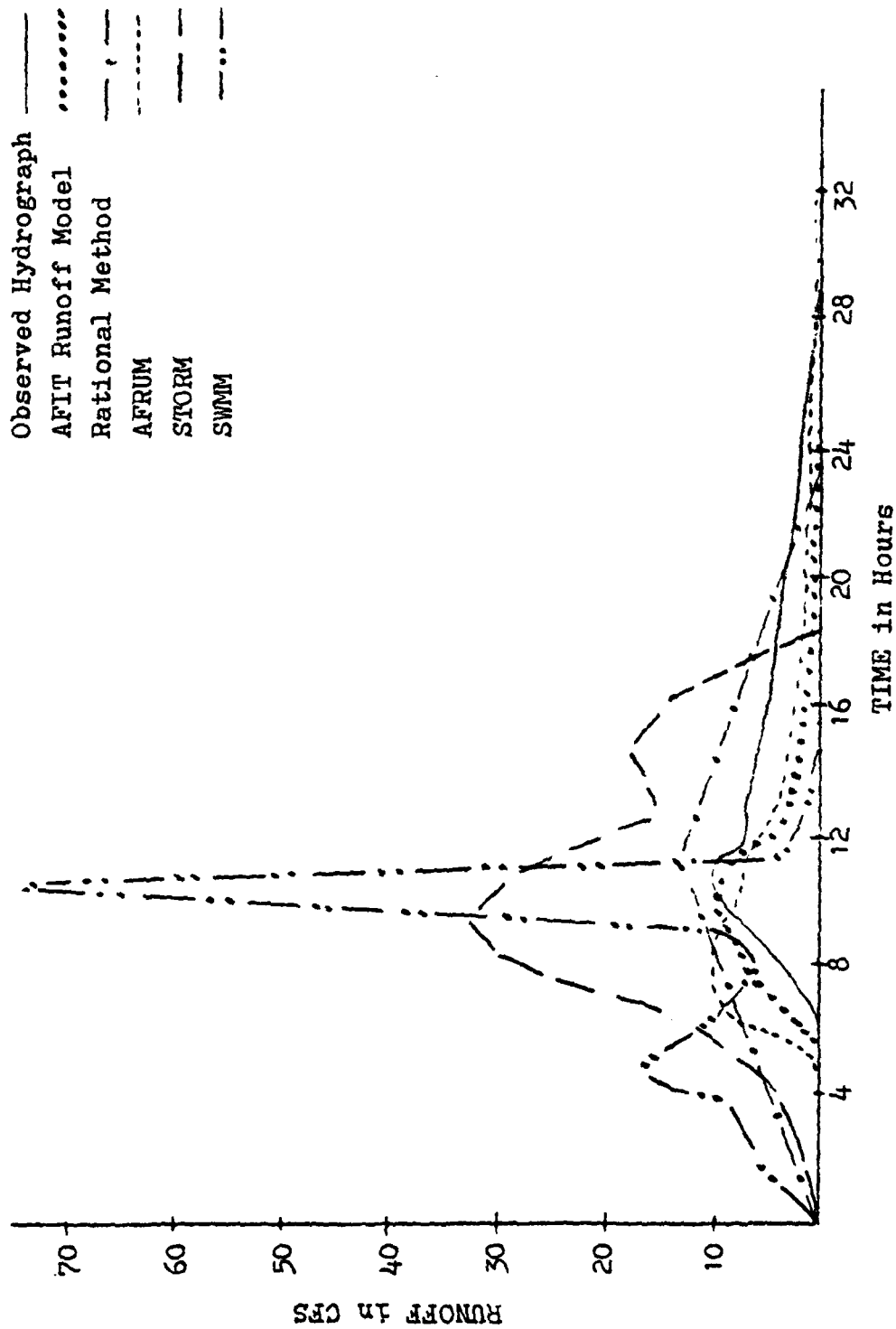
97

Observed Hydrograph ——
AFIT Runoff Model ••••••••
Rational Method ——
AFRUM ——
STORM ——
SWMM ——

RUNOFF in CFS

70
60
50
40
30
20
10

TIME in Hours

4   8   12   16   20   24   28   32

G-4   MCDOWELL DITCH   14 November 1978

98
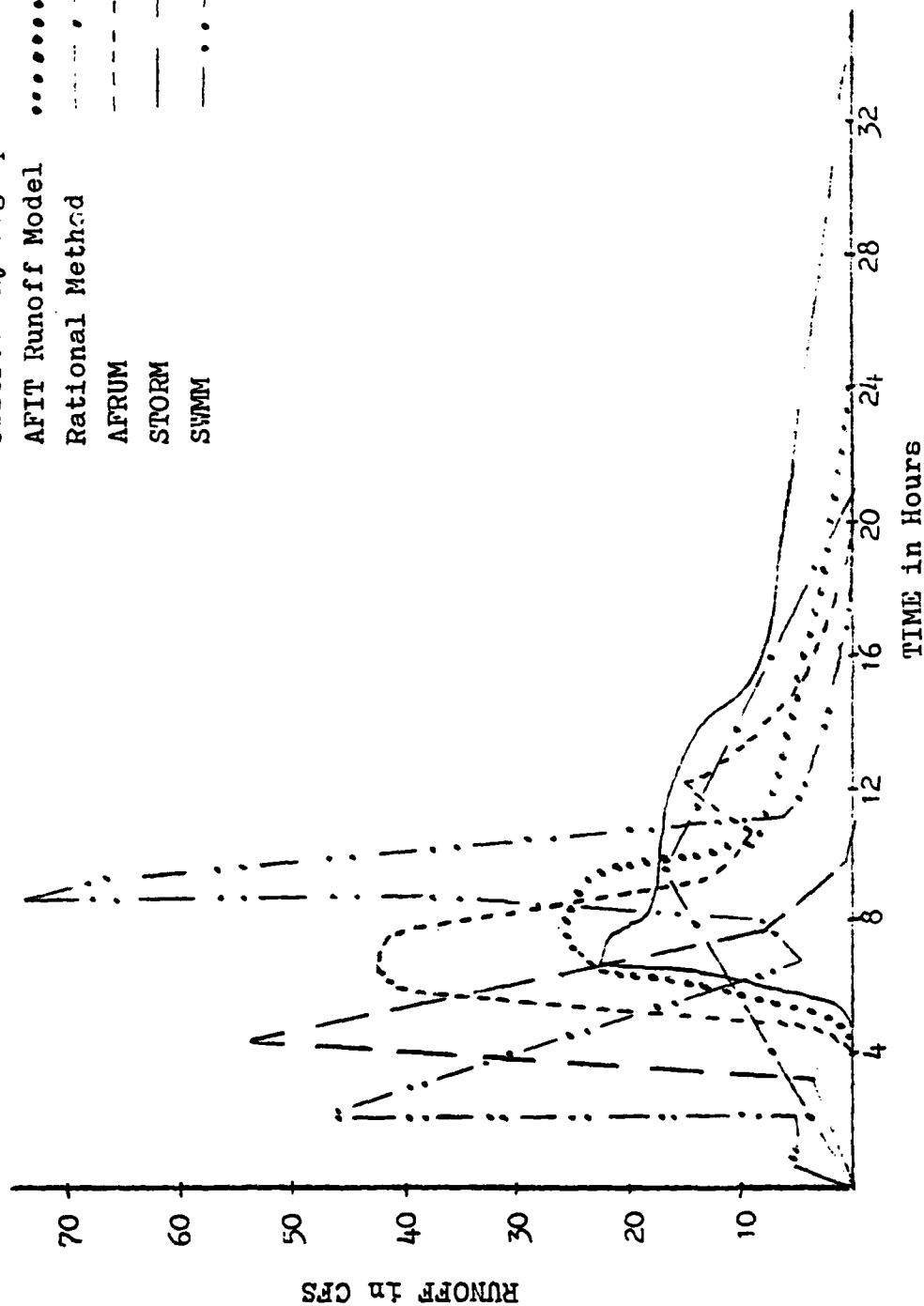
G-5 MCDOWELL DITCH 3 December 1978

Legend:
Observed Hydrograph
AFIT Runoff Model
Rational Method
AFRUM
STORM
SWMM

RUNOFF in CFS

TIME in Hours

G-6   MCDOWELL DITCH 11 April 1979

Legend:
Observed Hydrograph
AFIT Runoff Model
Rational Method
AFRUM
STORM
SWMM

Y-axis: RUNOFF in CFS
X-axis: TIME in Hours

100

Observed Hydrograph
AFIT Runoff Model
AFRUM
STORM
SWMM
Rational Method

Q_p = 225cfs
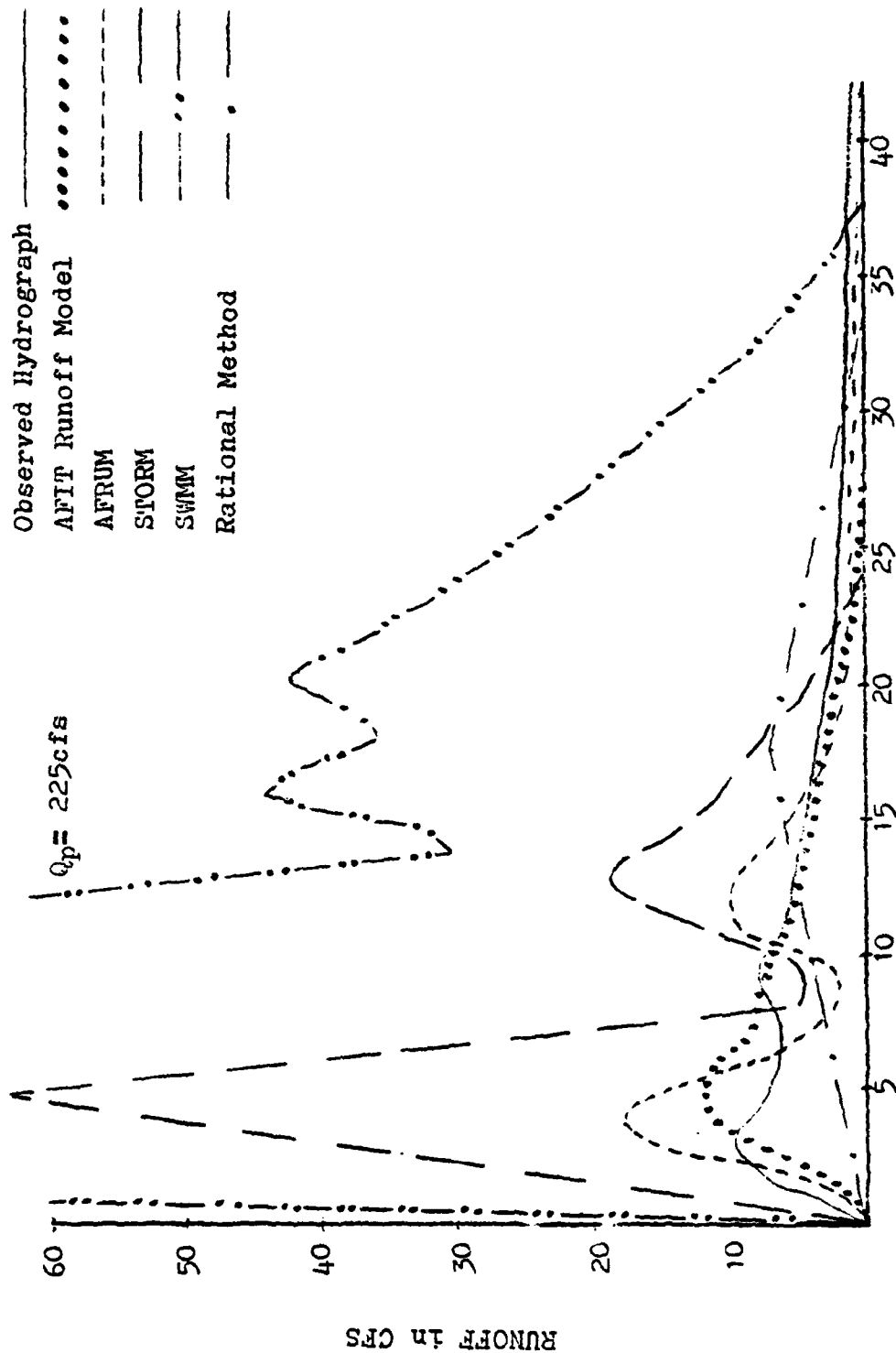
G-7   MCDOWELL DITCH 8 July 1979

TIME in Hours

RUNOFF in CFS

101

Observed Hydrograph
AFIT Runoff Model
Rational Method
AFRUM
STORM
SWMM

RUNOFF in CFS

TIME in Hours

G-8   EAST DITCH   14 November 1978

Legend:
- Observed Hydrograph
- AFIT Runoff Model
- Rational Method
- AFRUM
- STORM
- SWMM

RUNOFF in CFS
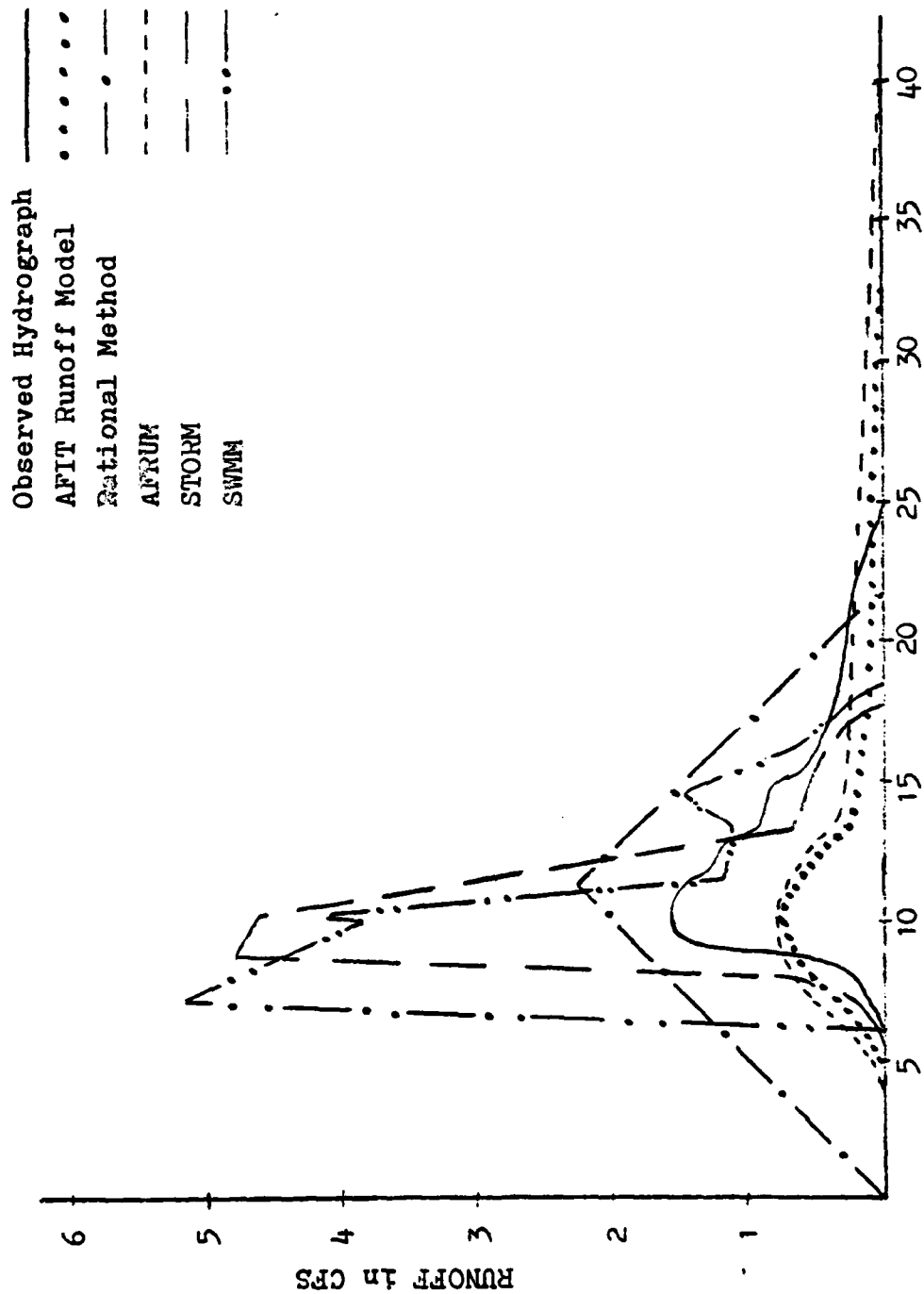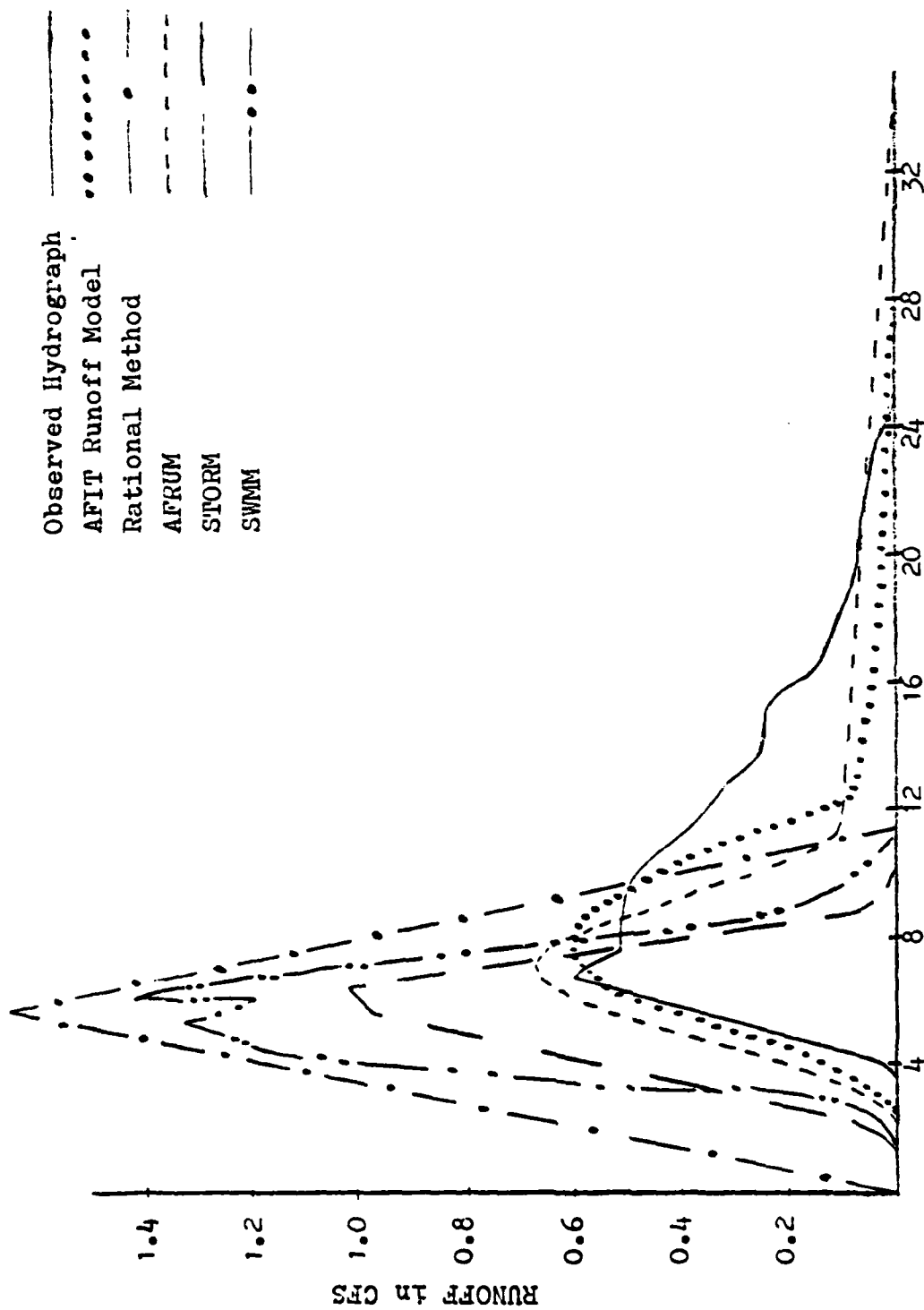
TIME in Hours

G-9   EAST DITCH   17 November 1978

Legend:
- Observed Hydrograph
- AFIT Runoff Model
- Rational Method
- AFRUM
- STORM
- SWMM

RUNOFF in CFS
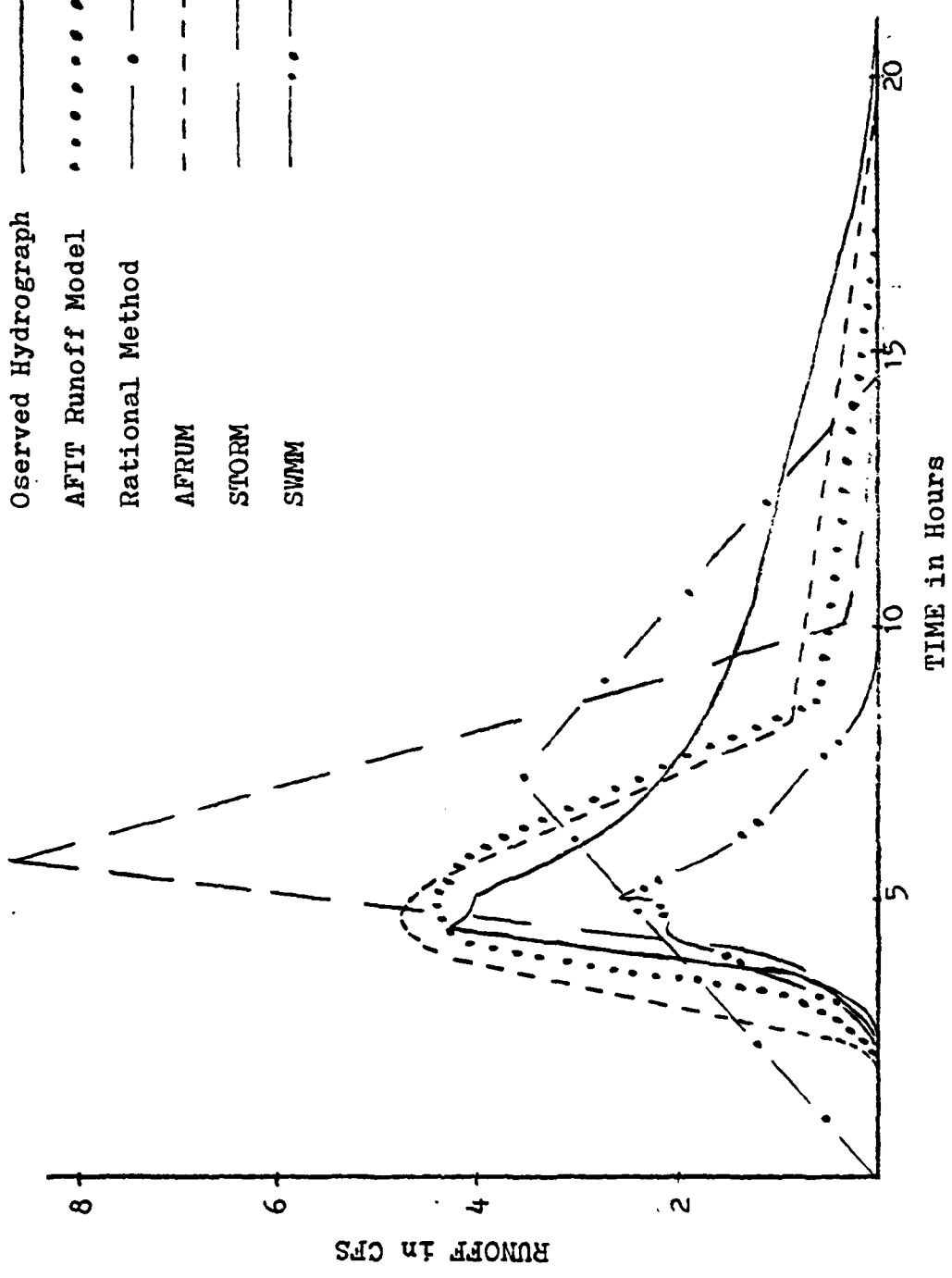
TIME in Hours

G-10   EAST DITCH   3 December 1978

Legend:
Observed Hydrograph
AFIT Runoff Model
Rational Method
AFRUM
STORM
SWMM

RUNOFF in CFS
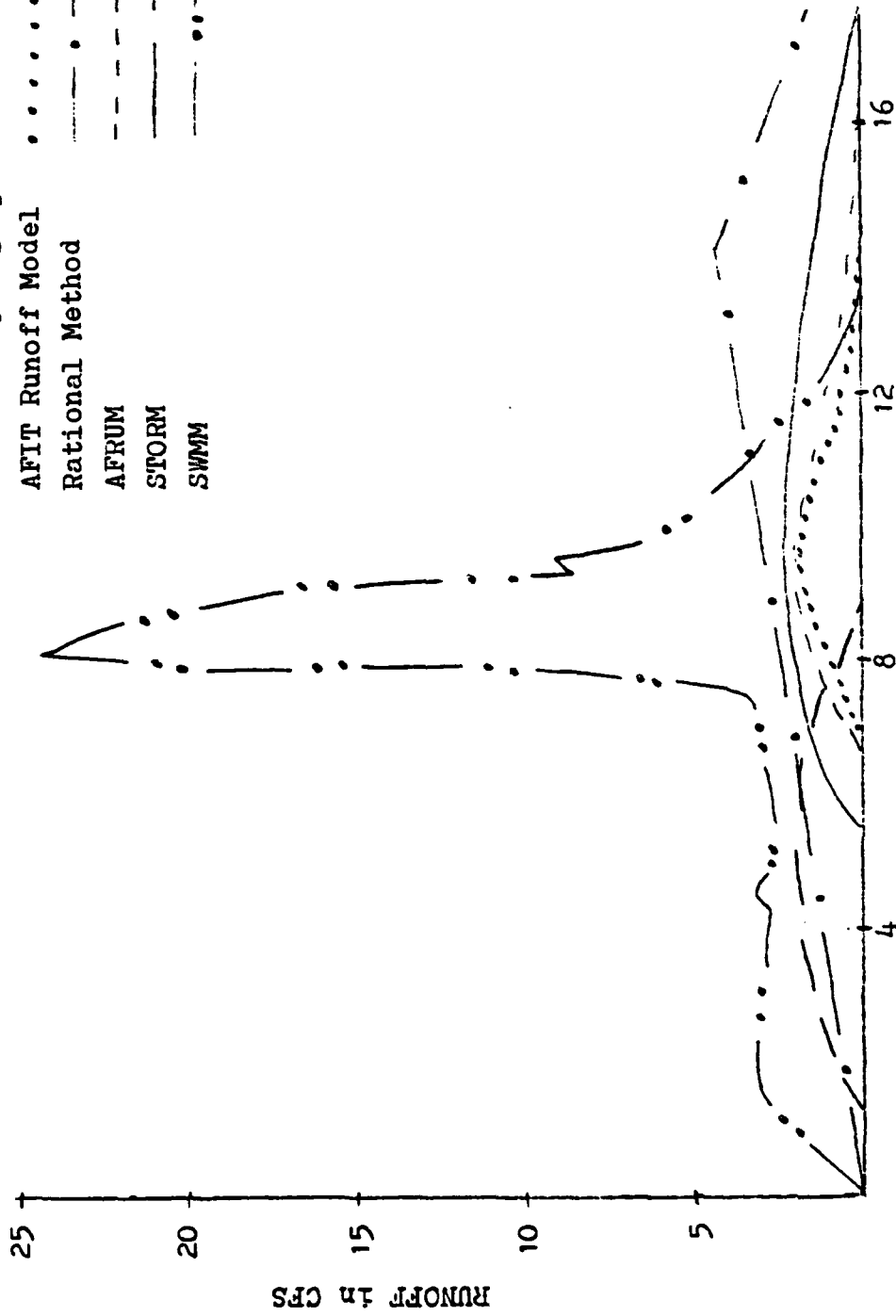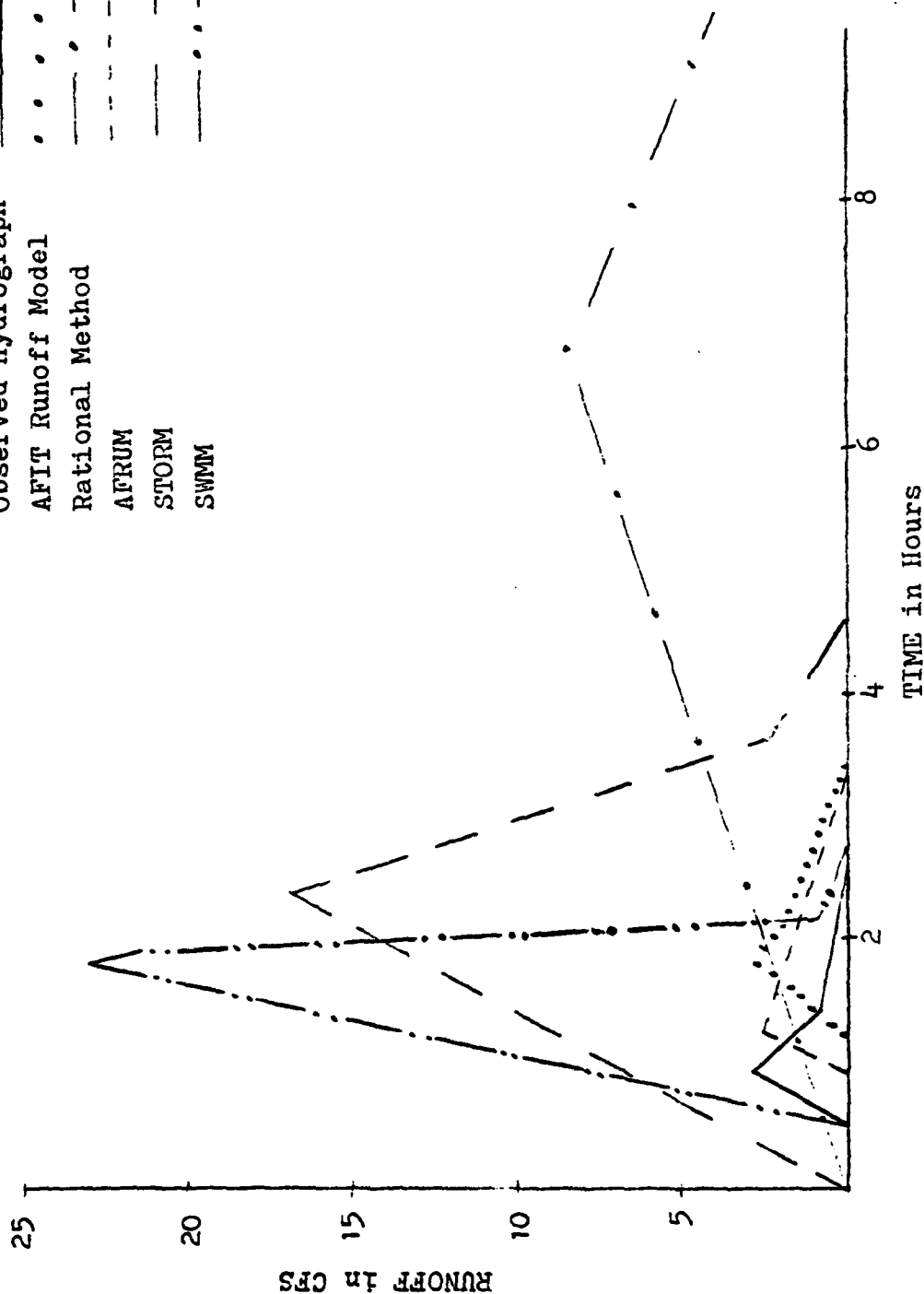
TIME in Hours

G-11   EAST DITCH   7 December 1978

105
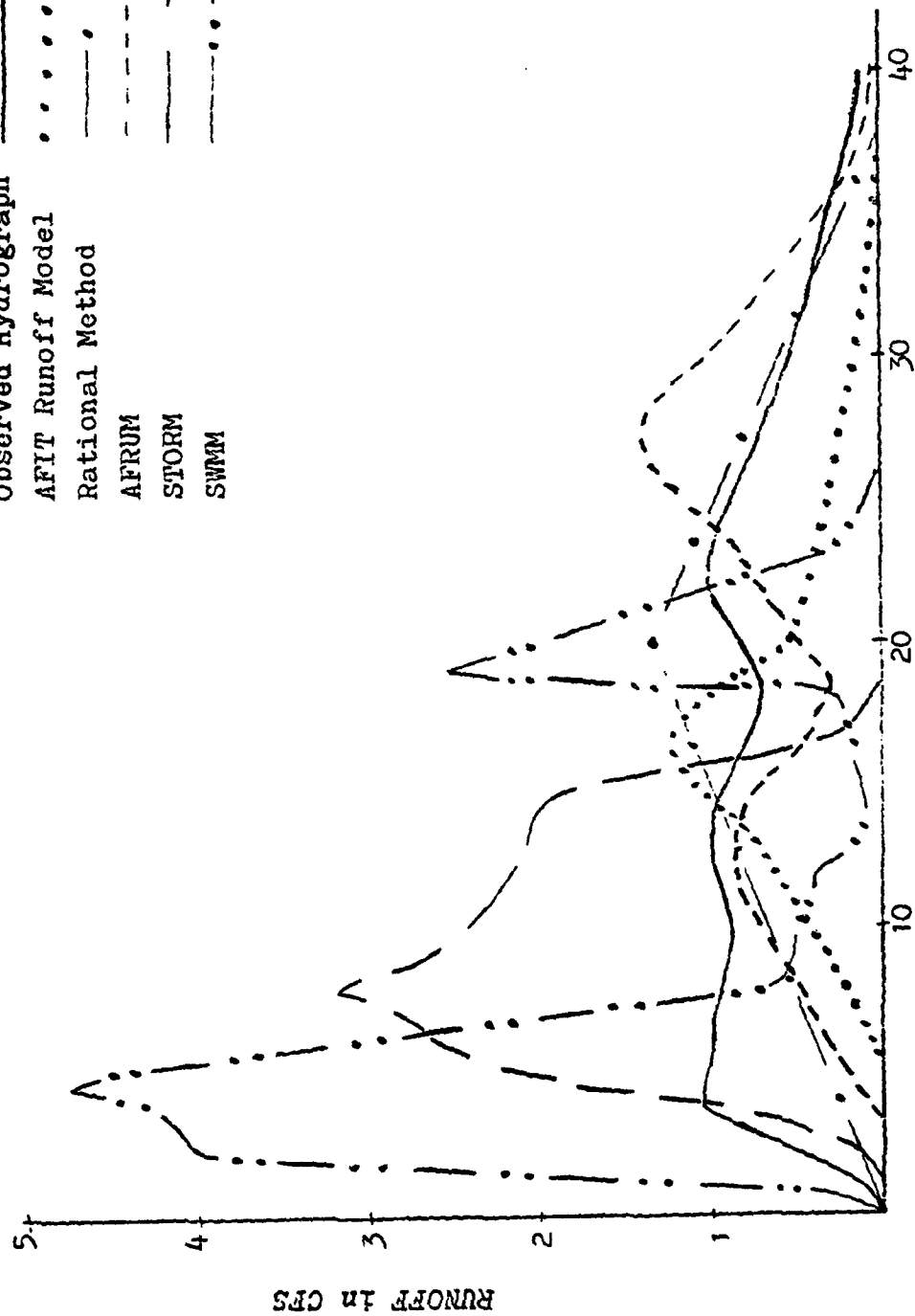
G-12   EAST DITCH   11 April 1979

Legend:
- Observed Hydrograph
- AFIT Runoff Model
- Rational Method
- AFRUM
- STORM
- SWMM

TIME in Hours

RUNOFF in CFS

106
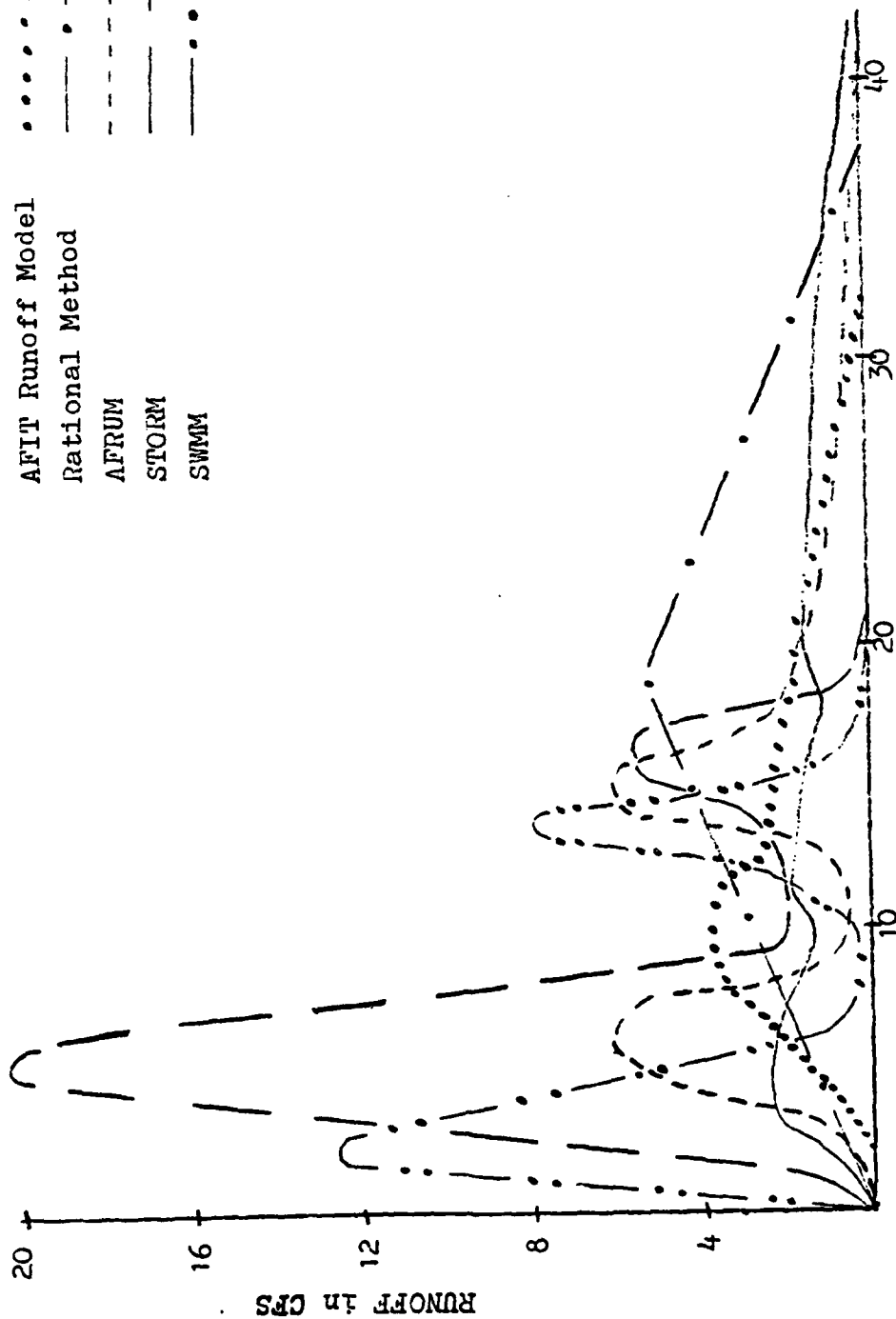
G-13   EAST DITCH   8 July 1979

Legend:
Observed Hydrograph
AFIT Runoff Model
Rational Method
AFRUM
STORM
SWMM

RUNOFF in CFS

TIME in Hours

G-14   CLINE DITCH   2 August 1978
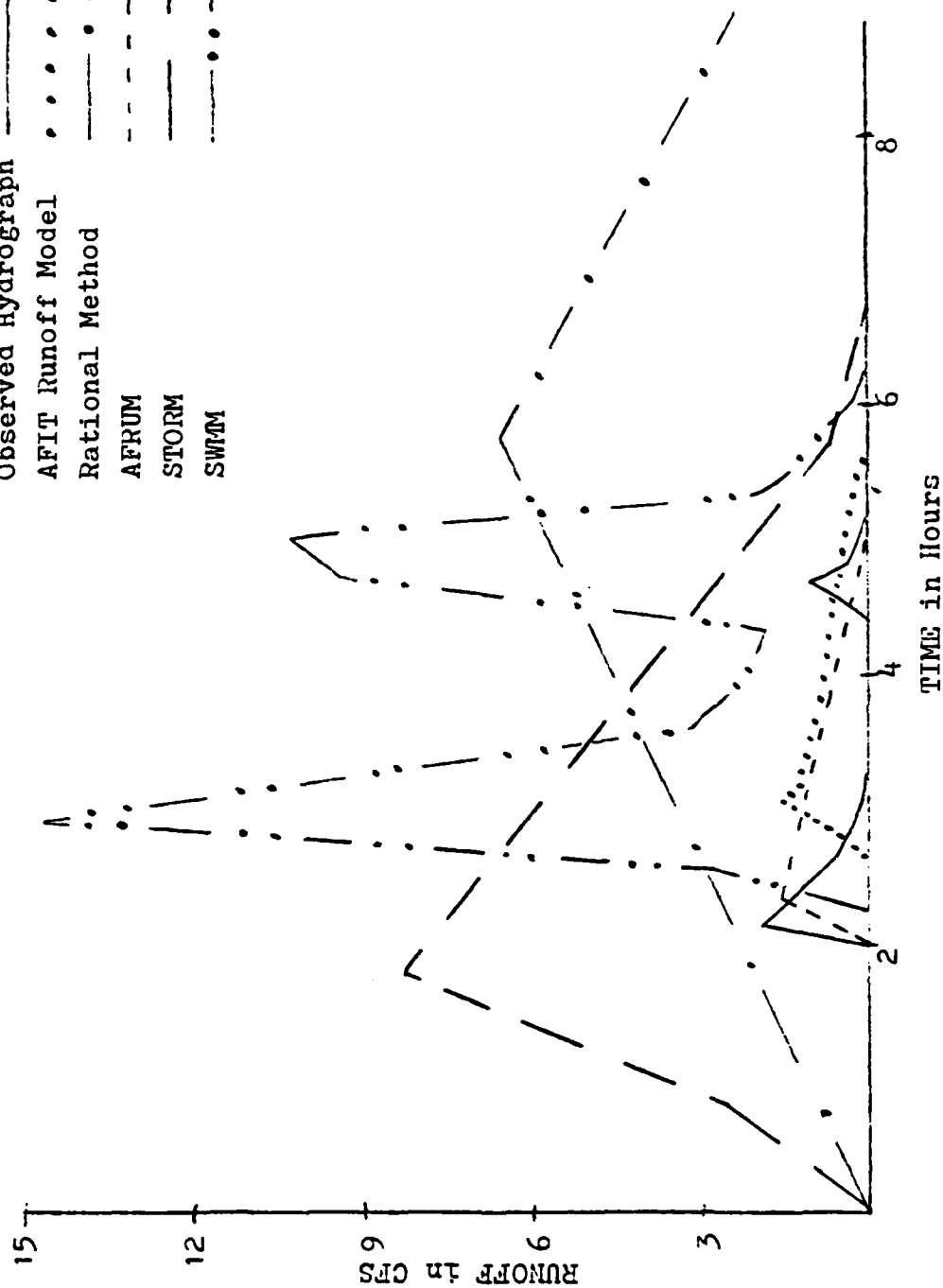
G-15 CLINE DITCH   27 August 1978

Legend:
Observed Hydrograph
AFIT Runoff Model
Rational Method
AFRUM
STORM
SWMM

RUNOFF in CFS

TIME in Hours

$Q_p = 32$ cfs

Observed Hydrograph
AFIT Runoff Model
Rational Method
AFRUM
STORM
SWMM

RUNOFF in CFS

TIME in Hours
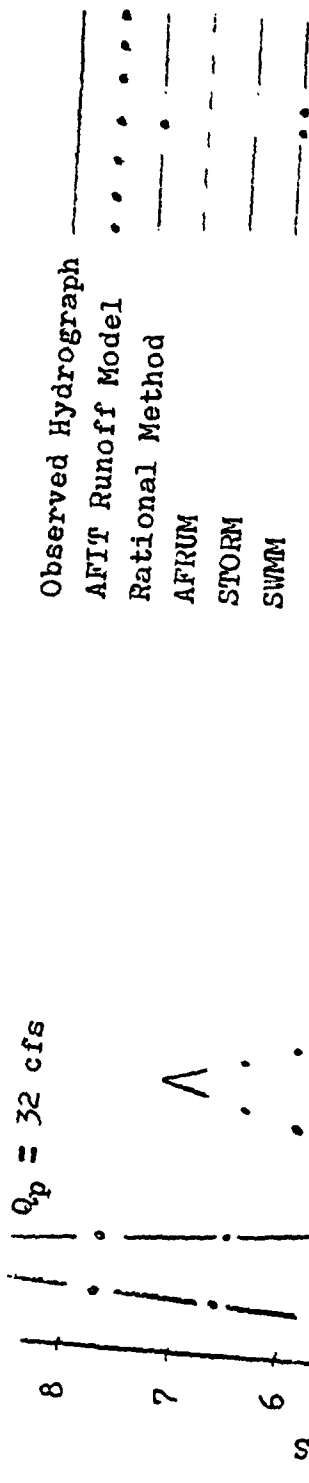
G-16   CLINE DITCH   14 September 1978

110

Legend:
- Observed Hydrograph
- AFIT Runoff Model
- Rational Method
- AFRUM
- STORM
- SWMM

RUNOFF in CFS

TIME in Hours

G-17   CLINE DITCH   3 December 1978
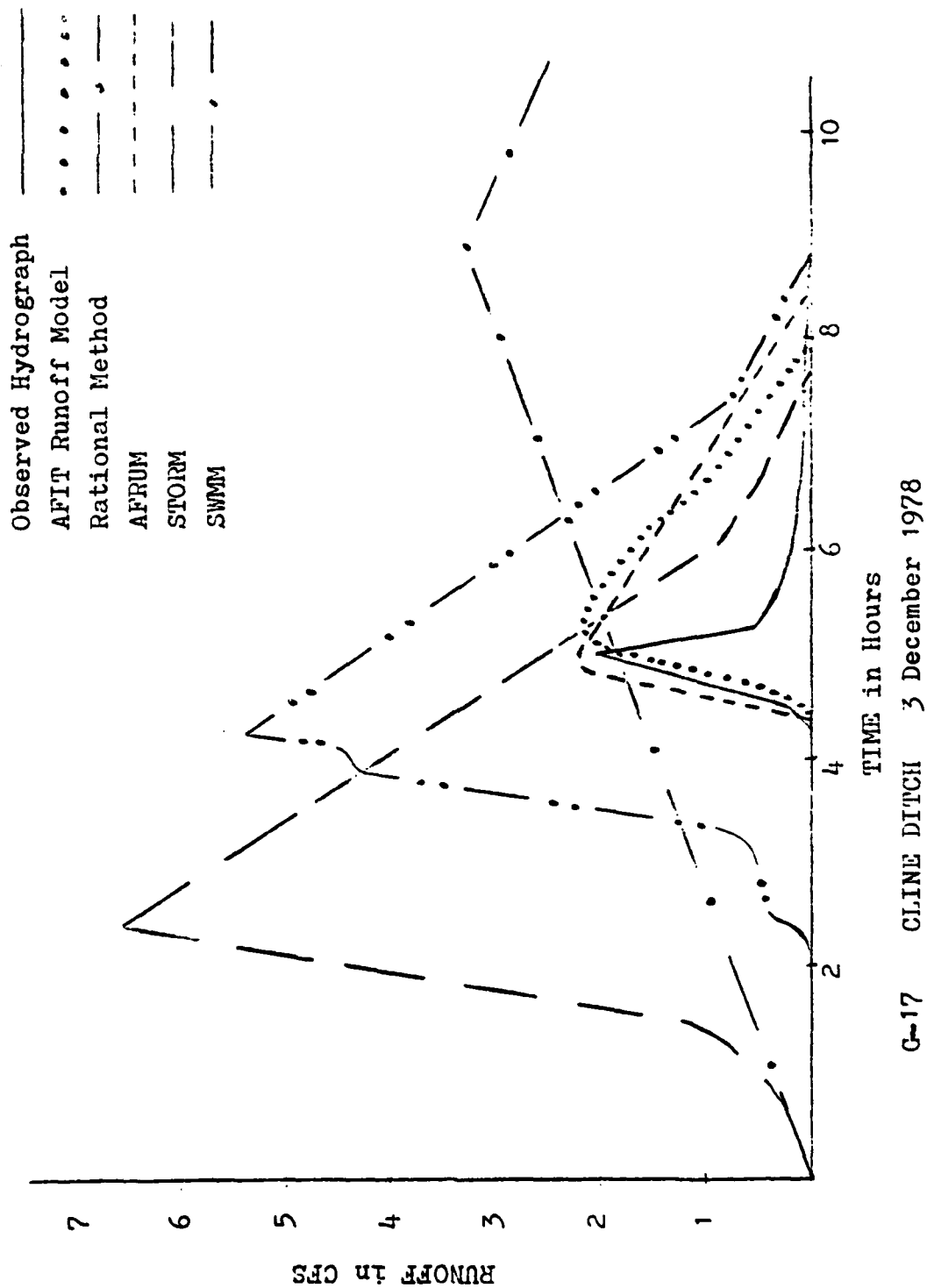
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